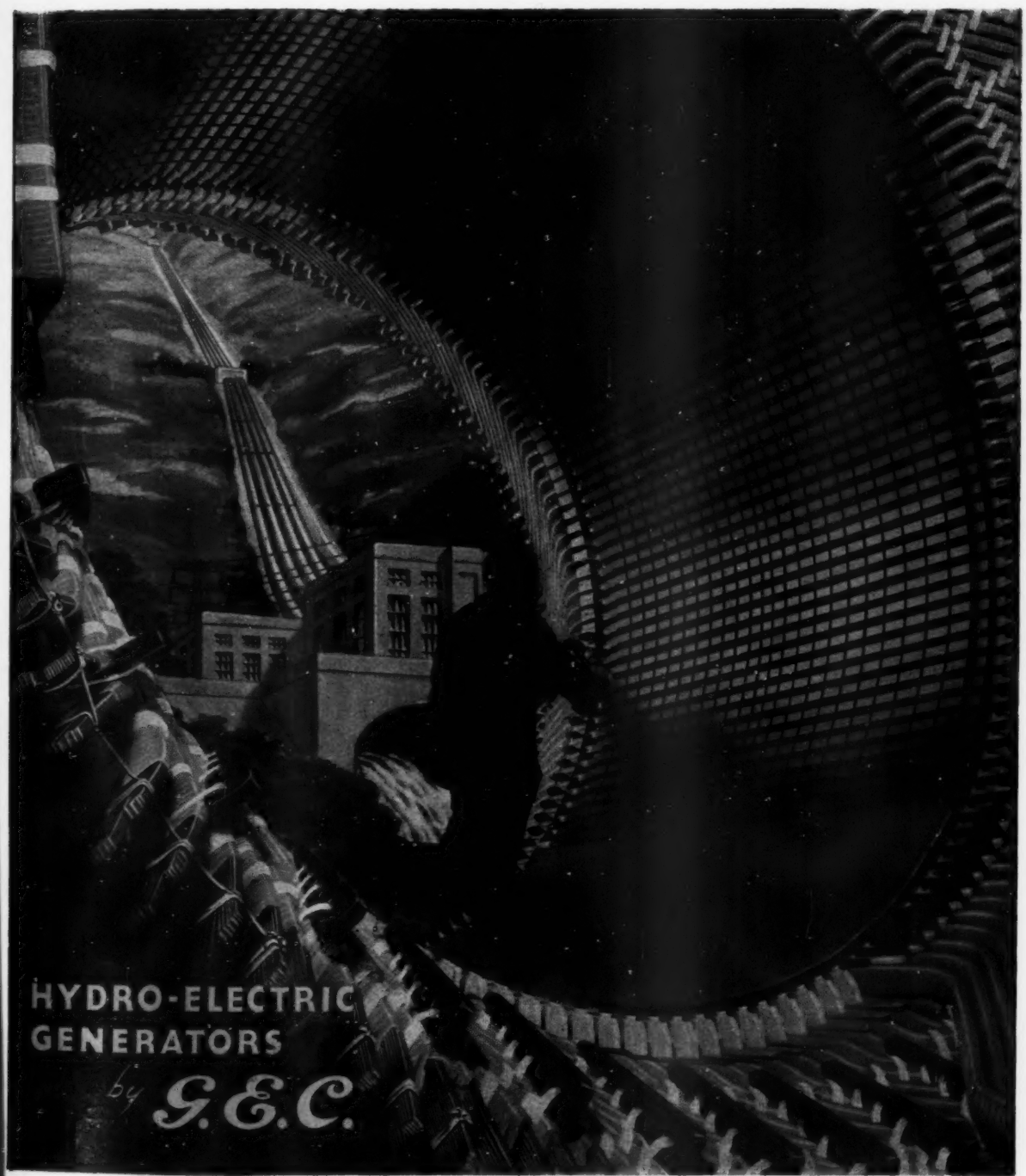
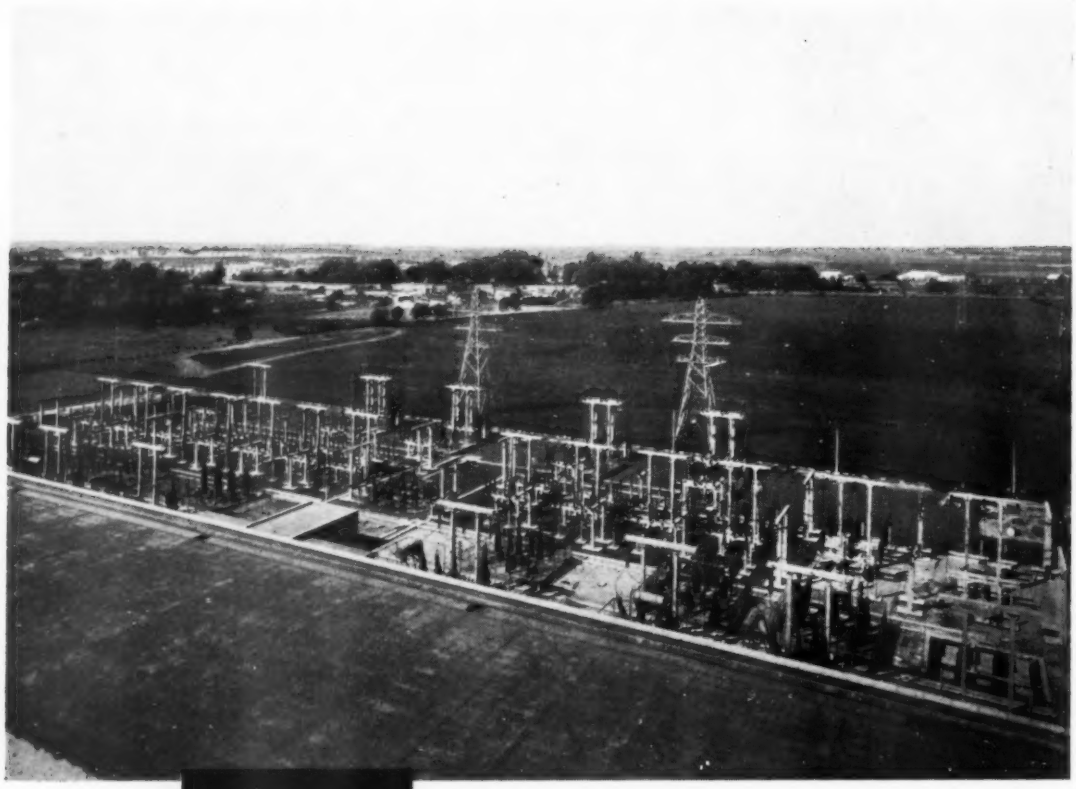


The General Electric Co. Ltd., Chief Office for Scotland: Magnet House, 71 Waterloo Street, Glasgow.
Edinburgh Branch: Magnet House, 8, George Street. Dundee Branch: 26-30 North Lindsay Street.
Aberdeen Branch: 7 Willowdale Place. Inverness Branch: 14 Falcon Square.



GOLDINGTON POWER STATION

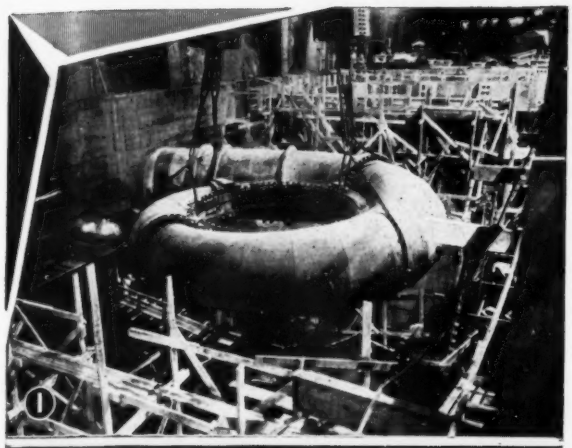
Consulting Engineers : Messrs. Merz and McLellan

Photograph by courtesy of the CEA Eastern Division

The main 132-kV 2500-MVA air-blast switchgear with its associated control-equipment as well as the 3·3-kV oil-break switchgear and 415-volt air-break switch and fuse gear for the station auxiliaries at Goldington Power Station was supplied by Reyrolle.

Reyrolle

A. REYROLLE & COMPANY LIMITED - HEBBURN - COUNTY DURHAM - ENGLAND



turbines for three hydro-electric schemes add over 2,000,000 h.p. to Canada's power potential

For three of the world's largest hydro-electric projects 26 water turbines of 'ENGLISH ELECTRIC' design are being supplied by English Electric Company of Canada Ltd. These machines are being manufactured by our Canadian associates, John Inglis Co. Limited, and will add to Canada's power potential a total of over 2,000,000 h.p. Of this over 1,000,000 h.p. is already in operation.

1 **BERSIMIS**—WORLD'S LARGEST HIGH HEAD TURBINES DEVELOP RECORD OUTPUT

Throughout the past year at Bersimis-Lac Casse Development 520,000 kW has regularly been supplied to Hydro Quebec's power system from four 'ENGLISH ELECTRIC' water turbines. On test these turbines have developed 200,000 h.p. under a net head of 840 feet, thus establishing a world record for hydro-electric units.

2 **NIAGARA**—OUTSTANDING SUCCESS OF WORLD'S FIRST DERIAZ REVERSIBLE PUMP/TURBINES

Six 'ENGLISH ELECTRIC' Deriaz reversible variable pitch pump/turbines are in commercial operation at the Sir Adam Beck-Niagara Pumping Generating Station operated by the Hydro-Electric Power Commission of Ontario. Operating under 83 feet head, each machine is rated at 45,500 h.p. as a turbine and as a pump will discharge 4,000 to 5,000 cubic feet of water per second under a variable head. These pump/turbines are of a new basic design developed by 'ENGLISH ELECTRIC' and have successfully absorbed 65,000 h.p. when pumping quantities in excess of 6,000 c.f.s. against a dynamic head of 80 feet.

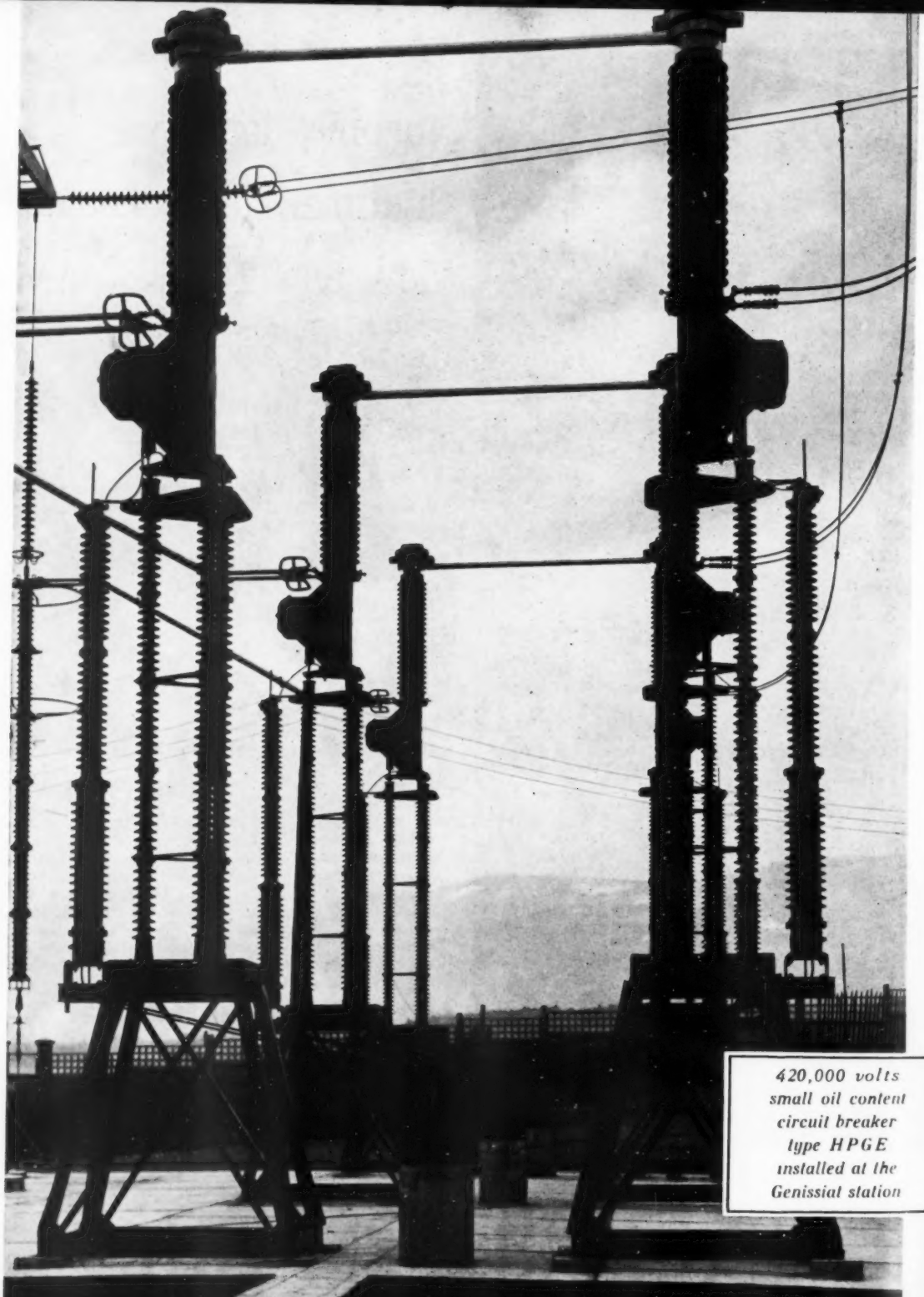
3 **ST. LAWRENCE POWER PROJECT**—SIXTEEN 75,000 h.p. TURBINES

For Ontario Hydro's Power Project on the St. Lawrence, sixteen 'ENGLISH ELECTRIC' 75,000 h.p. water turbines were ordered in 1955. The first four machines went into commercial operation in July, 1958. The power generated will be Canada's share of the output from this international power project in conjunction with the St. Lawrence Seaway.

'ENGLISH ELECTRIC'
water turbines

THE ENGLISH ELECTRIC COMPANY LIMITED, MARCONI HOUSE, STRAND, LONDON, W.C.2.
ENGLISH ELECTRIC COMPANY OF CANADA LIMITED, ST. CATHARINES, ONTARIO.
Offices and representatives throughout the world.

HE. 32 H8



420,000 volts small oil content circuit breaker type HPGE installed at the Genissiat station

THE WHOLE OF APPARATUS UP TO 500,000 VOLTS

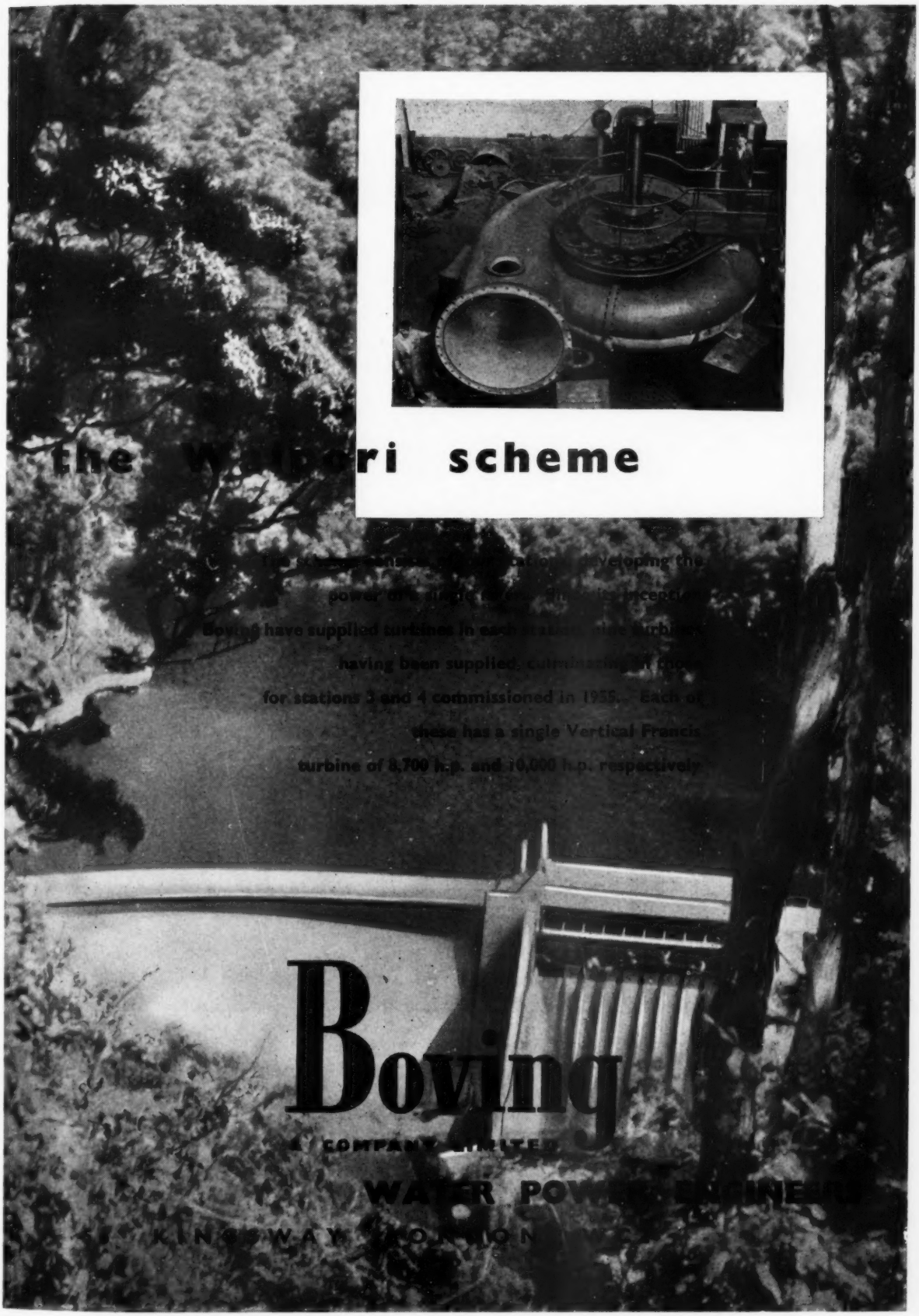
Ateliers de Constructions Electriques de

DELLE

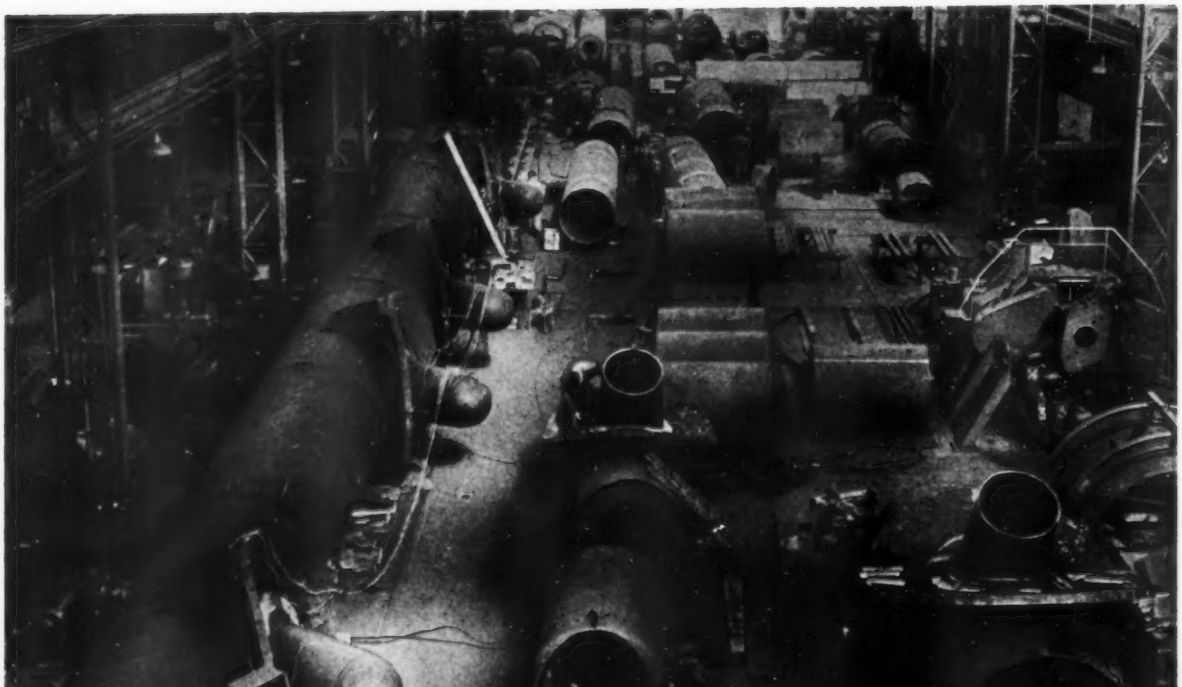
DE LA COMPAGNIE GÉNÉRALE D'ELECTRICITÉ

25, CHEMIN DE CYPRIAN · VILLEURBANNE · FRANCE

9943


the W ri scheme
power of a single
Boving have supplied turbines in each
having been supplied, culminating
for stations 3 and 4 commissioned in 1955. Each of
these has a single Vertical Francis
turbine of 8,700 h.p. and 10,000 h.p. respectively
Boving
COMPANY LIMITED
WATER POW ENGINEERS
KINGSWAY LONDON

Penstocks, pressure shafts linings,
Substation steel structures
Gantry and overhead travelling cranes
Pylons, water gates



Downstream portion of the Grande Dixence Fionnay Power Station distribution pipe during tensometric shop tests. Max. wall thickness 52 mm., service pressure 96 at., test pressure 121 at.

GIOVANOLA **Monthey - Switzerland**

England – France

DUNGENESS – a new power alliance

DOVER

CALAIS

BOULOGNE

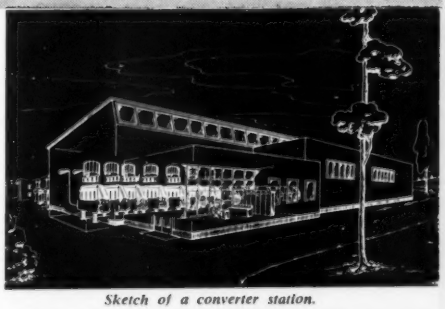
The French and British power administrations, Electricité de France and the Central Electricity Generating Board, have now decided to carry into effect the plan for the exchange of power across the English Channel, which has been under discussion ever since the end of the Second World War.

At the end of 1957 the system developed and proposed by ASEA for high-voltage direct current, which had already been adopted for the transmission of power from the Swedish mainland to the island of Gotland in 1954—the first and only project of its kind in the world—was officially accepted. The definite contract has recently been awarded to ASEA for the principal electrical equipment for the transmission in either direction of 160,000 kW, equivalent to 800 A d. c. at 200 kV. It is anticipated that the installation will be ready for commissioning by the end of 1960.

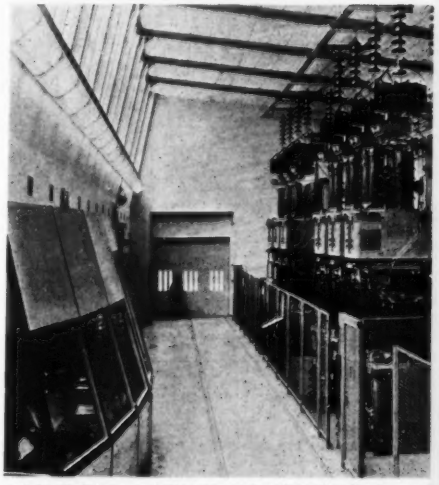
The idea behind this project is to utilise surplus power from the one country in the other country during periods of peak load. It so happens that the load peaks occur at different times of the day and during different days of the week. Despite the close proximity of the two countries, there are also certain seasonal variations in the load peaks. In addition, this project will enable both countries to postpone or even refrain from building new power stations, which would entail a considerably greater capital expenditure than the planned d. c. link.

The transmission distance will be about 60 km. The converter stations are to be set up in the vicinity of Dungeness in England and Boulogne in France. It is intended to connect the d. c. link to the English 275 kV, 50 c/s grid, and to the French 225 kV, 50 c/s grid.

The ASEA d. c. system, which was selected in the face of severe competition from other alternative solutions, offers considerable advantages. The number of transmission cables can be limited, the installation costs are lower and the likelihood of mechanical faults less. Furthermore, the d. c. link will enable the network frequency of the one country to be regulated independently of that in the other country.



Sketch of a converter station.



Valve hall in one of the converter stations for the d.c. transmission system to Gotland, in operation since 1954.

Västerås

Sweden

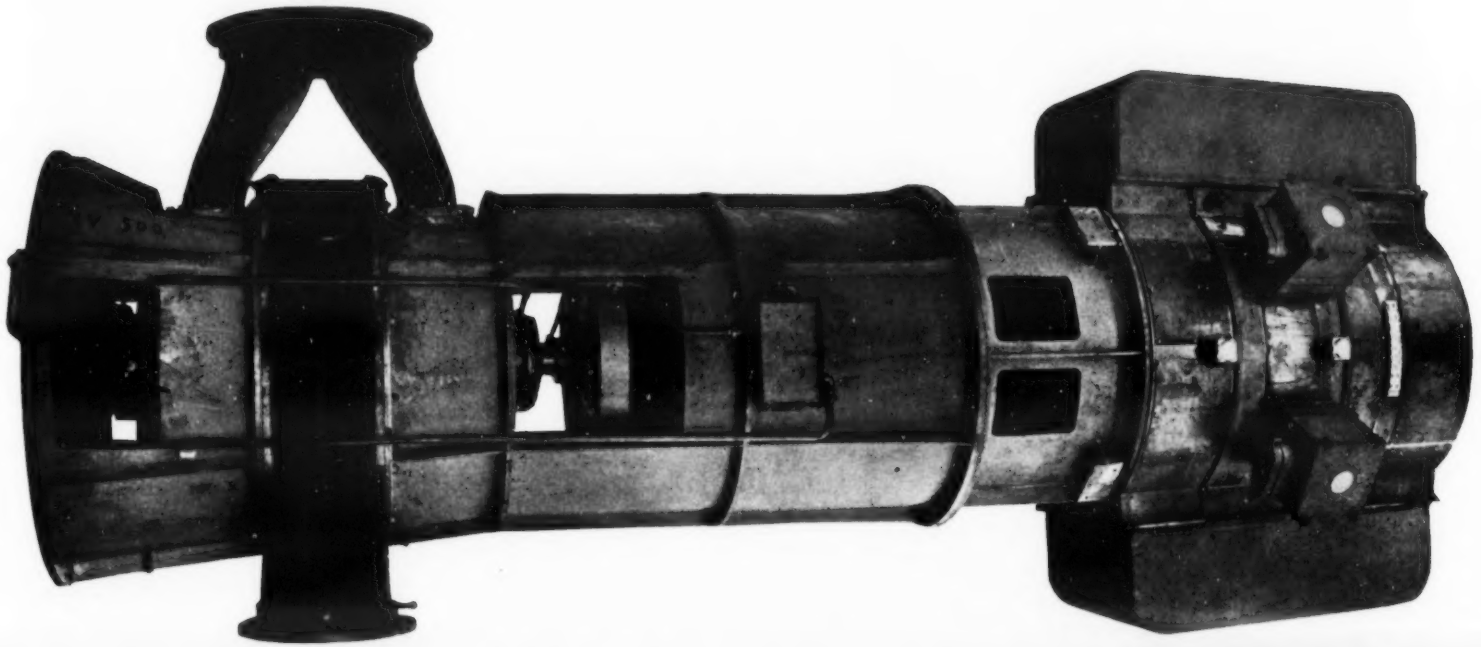
PELLIZZARI

ARZIGNANO ITALY

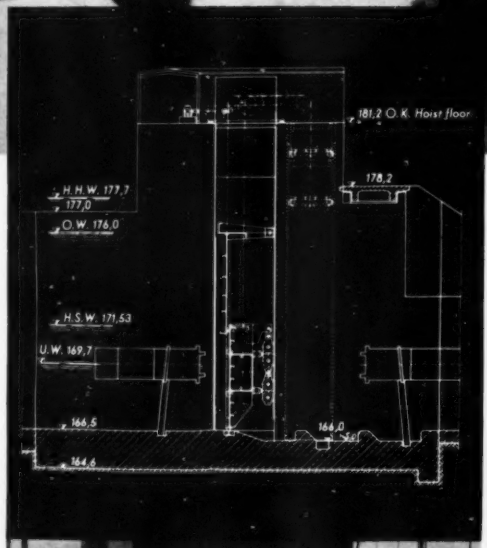
MINISTRY OF COMMUNICATIONS AND PUBLIC WORKS ATHENS (GREECE) VILIZA PUMPING STATION

ELECTRIC AND HYDRAULIC MACHINES

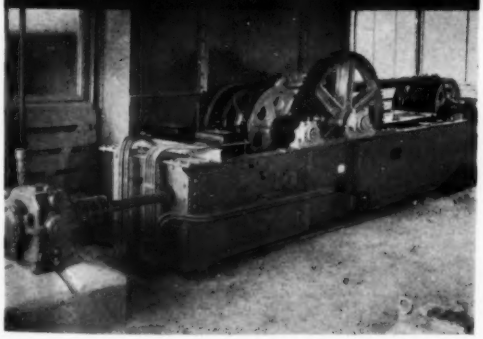
Pump Type AV 500 Capacity lt/sec. 1100 Total Man. Head mt 85
Absorbed Power h.p. 1450 Rated Speed r.p.m. 980
Driven by Electric Motor Type GASPV 1500/6 Rated Output h.p. 1700
Three phase Voltage Volt 3000 Frequency HZ 50
Rated Speed r.p.m. 980



Downstream gate of Besigheim lock two-part lifting gate with counterweight compensation



Cross section



Hoist

We construct and supply:

Hydraulic steel structures
Weirs
River and sea locks
By-pass gates, safety gates
Valley storage dams
High-pressure piping (penstocks)
Dock gates
Ship lifts
Slipways
Floating cranes

All hydraulic structures (including mechanical and electrical equipment) are supplied and erected ready for operation.

FRIED. KRUPP MASCHINEN- UND STAHLBAU RHEINHAUSEN

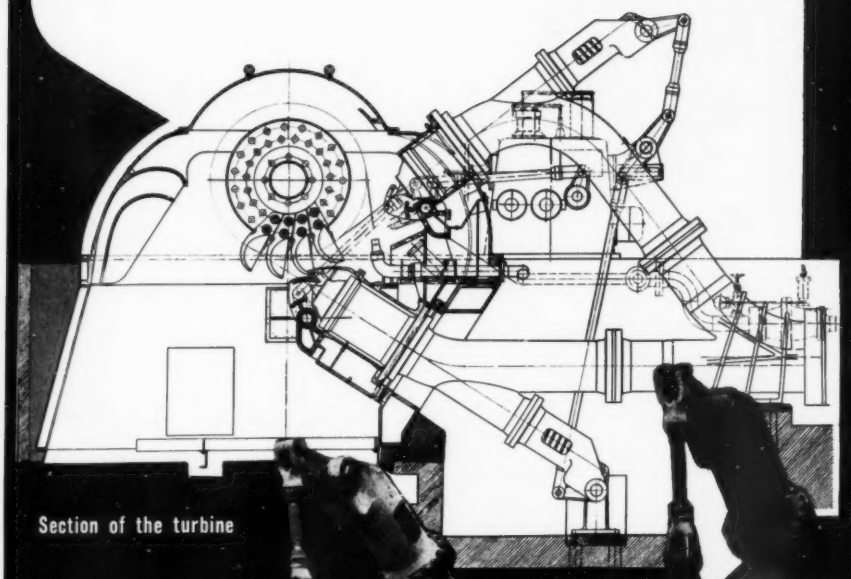
PIRELLI GENERAL

SOUTHAMPTON

cables

Pelton-Turbines

AS DOUBLE TURBINES WITH 4 JETS UP TO THE MAXIMUM OUTPUTS INCLUDING REGULATING EQUIPMENT AND SHUT-OFF DEVICES



Section of the turbine

Double turbine with 4 jets output 36,000 h.p.

J.M. VOITH AUSTRIA

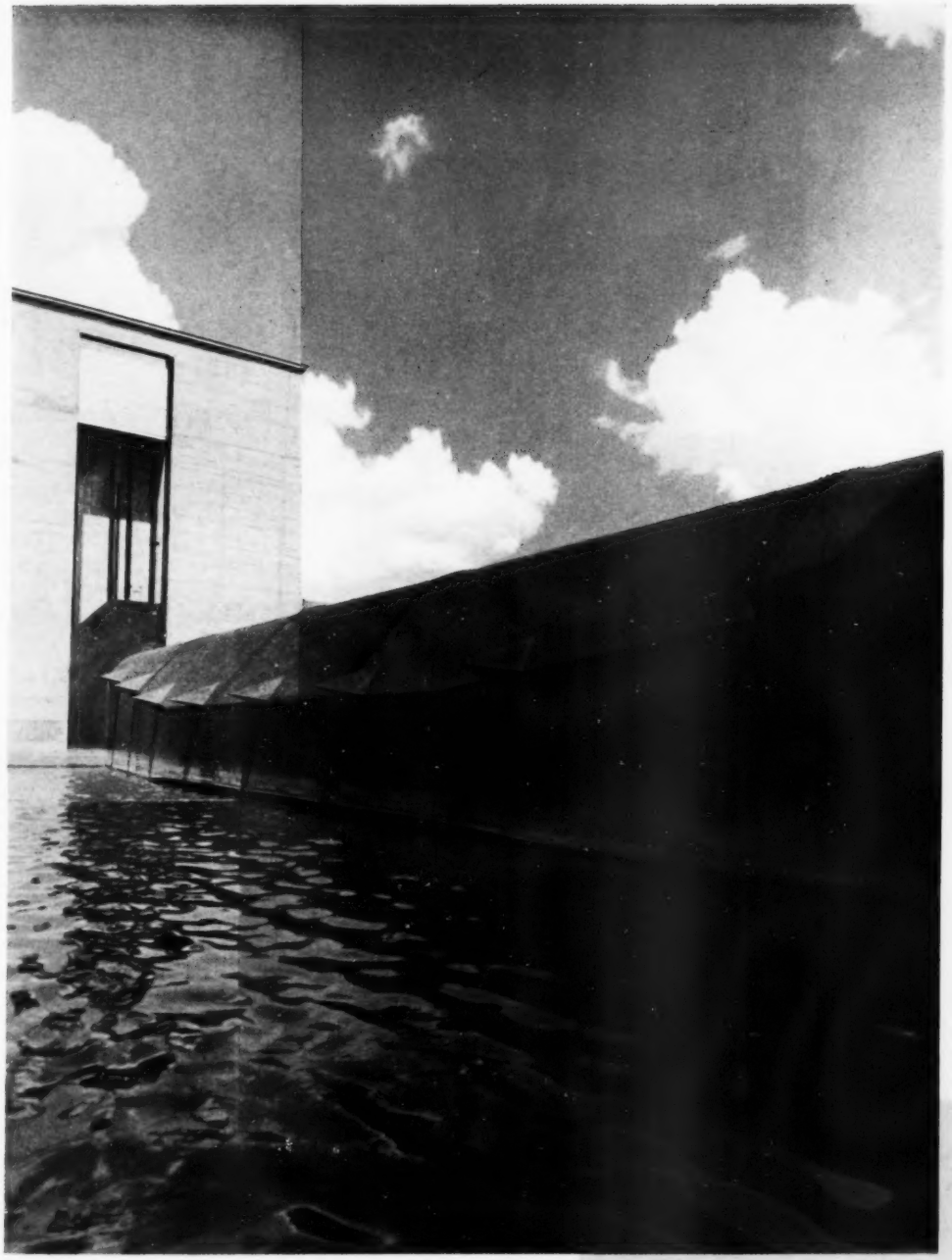
ST. POELTEN P.O.B. 168 TELEPHONE 2501

TELETYPE 01 2148 TELEGRAM ADDRESS: VOITHWERK ST. POELTEN

TU 674e

the Steel Foundry for all Water Turbine Castings

M·A·N

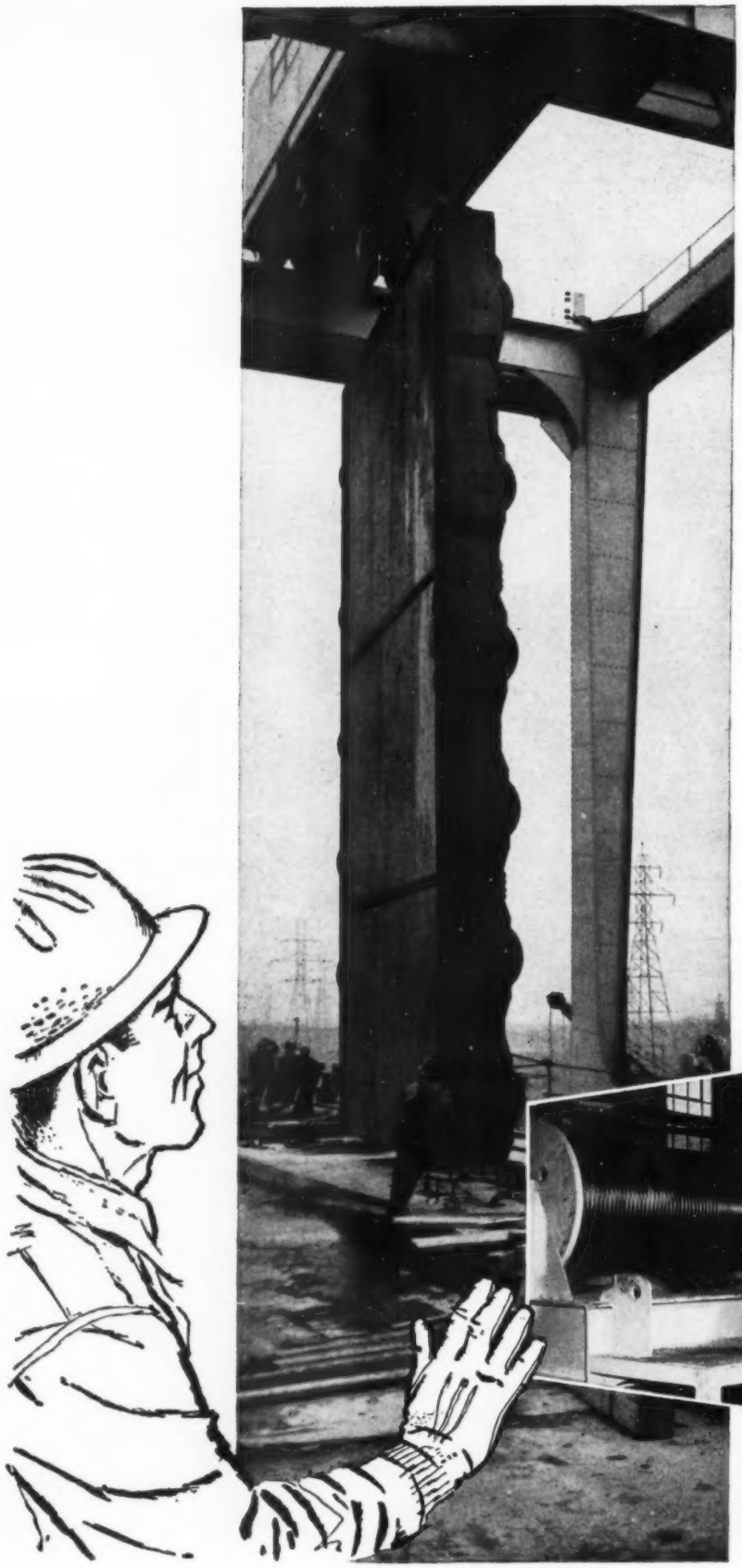


In hydro-power plants, minute regulation of the water level is an important feature best achieved by discharging the upstream water over the gate. The M.A.N. submersible sluice gate is particularly suited to that requirement. Illustration shows sluice gate of 30 m clear width and 4.85 m water head supplied for Kitzingen Weir Plant / River Main. Gate was designed to be lowered 1.3 m.

YDRAULIC STRUCTURES

ASCHINENFABRIK AUGSBURG-NÜRNBERG AG · GUSTAVSBURG WORKS

USTAVSBURG/HESSEN (GERMANY) REPRESENTATIVES IN ALL COUNTRIES



Intake Gates

Illustrated is a completely assembled Intake Gate being lowered into position. This is one of twelve built by Canadian Vickers for the Pumped Storage Scheme part of the Sir Adam Beck No. 2 Development of the Hydro Electric Power Commission of Ontario, at Queenstown.

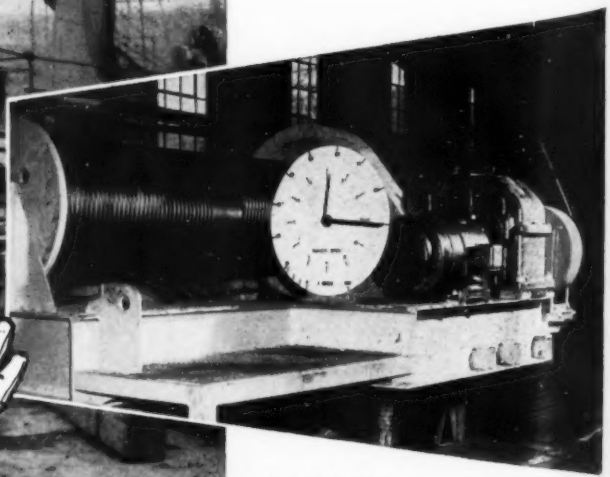
Shop welded in sections and riveted together in the field, these Gates are designed for a clear opening of 16′-8¾″ wide and 34′-9″ high and subjected to a maximum head on the sill of 77′-6″.

Canadian Vickers designs and builds all types of Water-Way and Water-Power equipment. If it is Vickers-built . . . it's right.

CANADIAN VICKERS LIMITED

MONTREAL TORONTO

CANADIAN MEMBER OF THE VICKERS GROUP



Shop photo of a Vickers-built Hoist.

2,340,000 kVA
of FERRANTI TRANSFORMERS
for the NIAGARA POWER PROJECT
LEWISTON, U.S.A.
FERRANTI
Ferranti Ltd. have been awarded a contract by the Power Authority of the State of New York to supply thirteen generator transformers for the Lewiston Power Station of the Niagara Power Project situated on the plateau on the Niagara River about 3 miles below the Falls in the town of Lewiston.
The contract covers six 180,000 kVA, 13.2/240 kV 3-phase transformers and seven 180,000 kVA, 13.2/120 kV 3-phase transformers. They will be of the forced-oil, forced-water cooled, inert gas-filled type and each will be directly coupled to a 167,000 kVA 3-phase generator.
Believed to be the largest transformer contract so far placed in Britain by the U.S.A., delivery will commence in the middle of 1959 and will be completed by the end of 1960.
F
FERRANTI
FERRANTI LTD · HOLLINWOOD · LANCS.
London Office: KERN HOUSE · 36 KINGSWAY · W.C.2

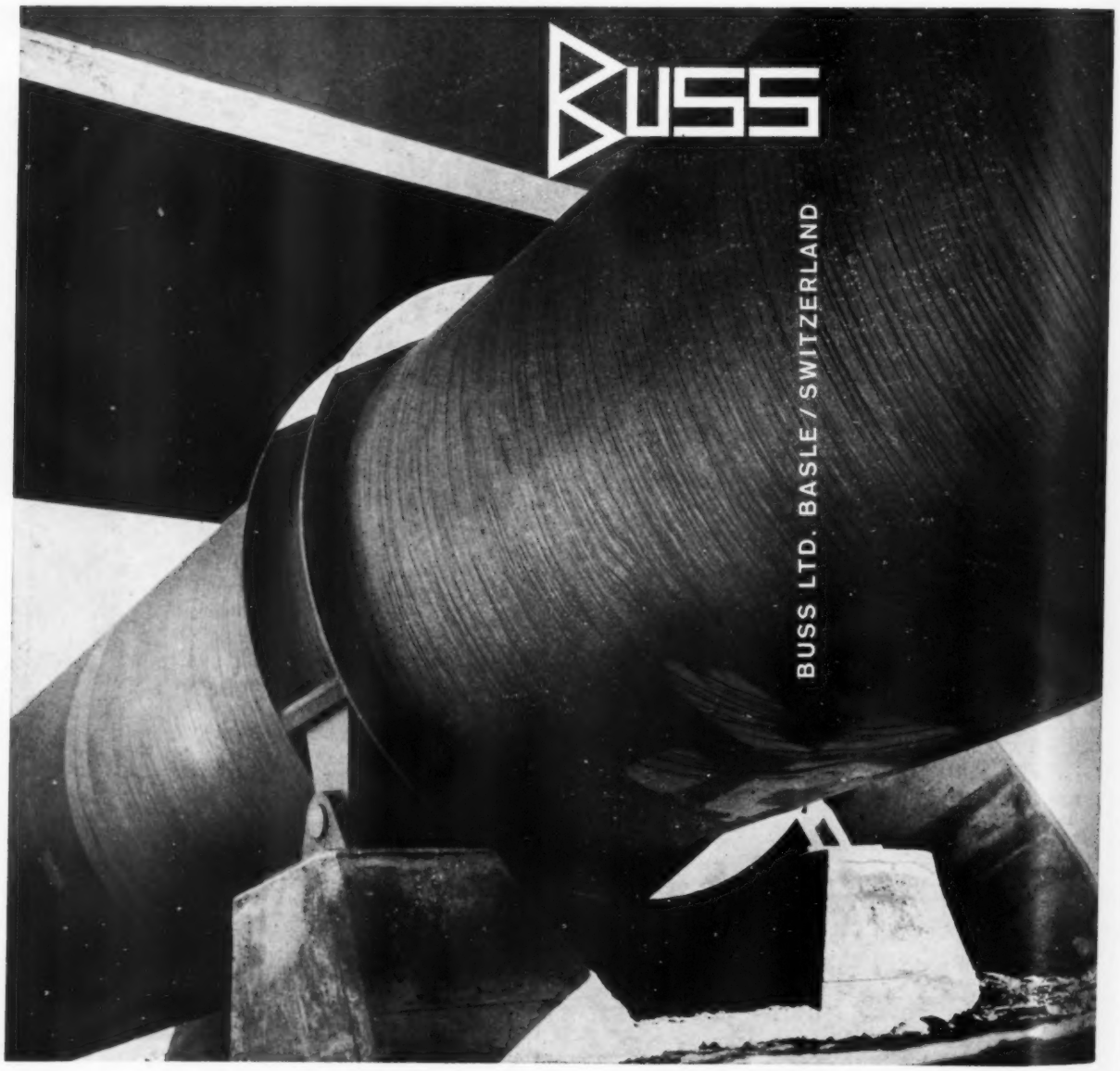
FT 216/2

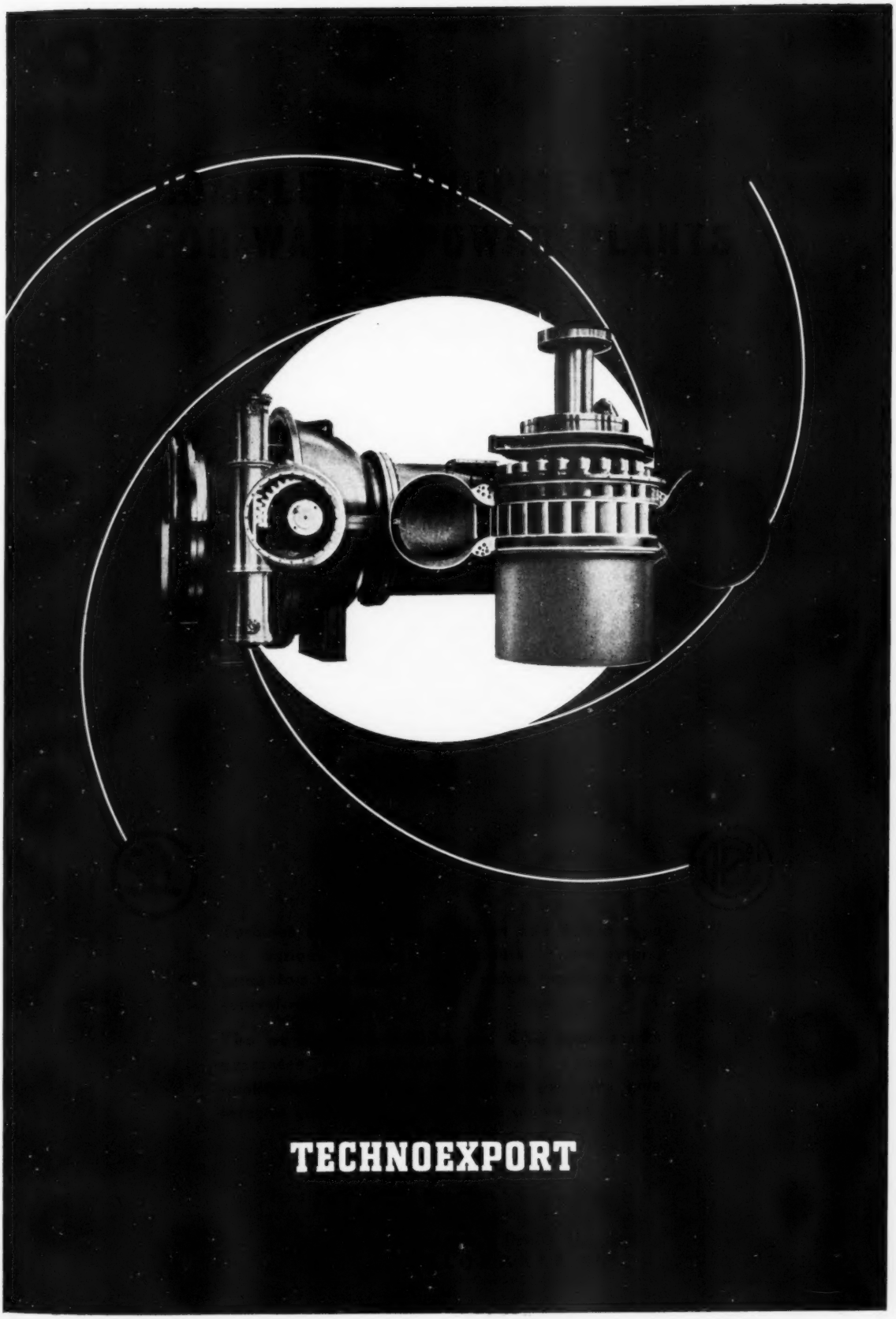
In the past few years we have manufactured and erected pipe-lines and tunnel reinforcements for the following hydraulic schemes:

Moyopampa (Peru)
Calancasca (Switzerland)
Kaftoun (Lybanon)
Châtelot (Switzerland)
Patla (Mexico)
Cavergno (Maggia, Switzerland)
Zongo (Belgian Congo)
Safien (Switzerland)
Vissoie (Switzerland)

The following are still in execution:

Kiymbi (Belgian Congo)
Pontirone (Switzerland)
Göschenen (Switzerland)

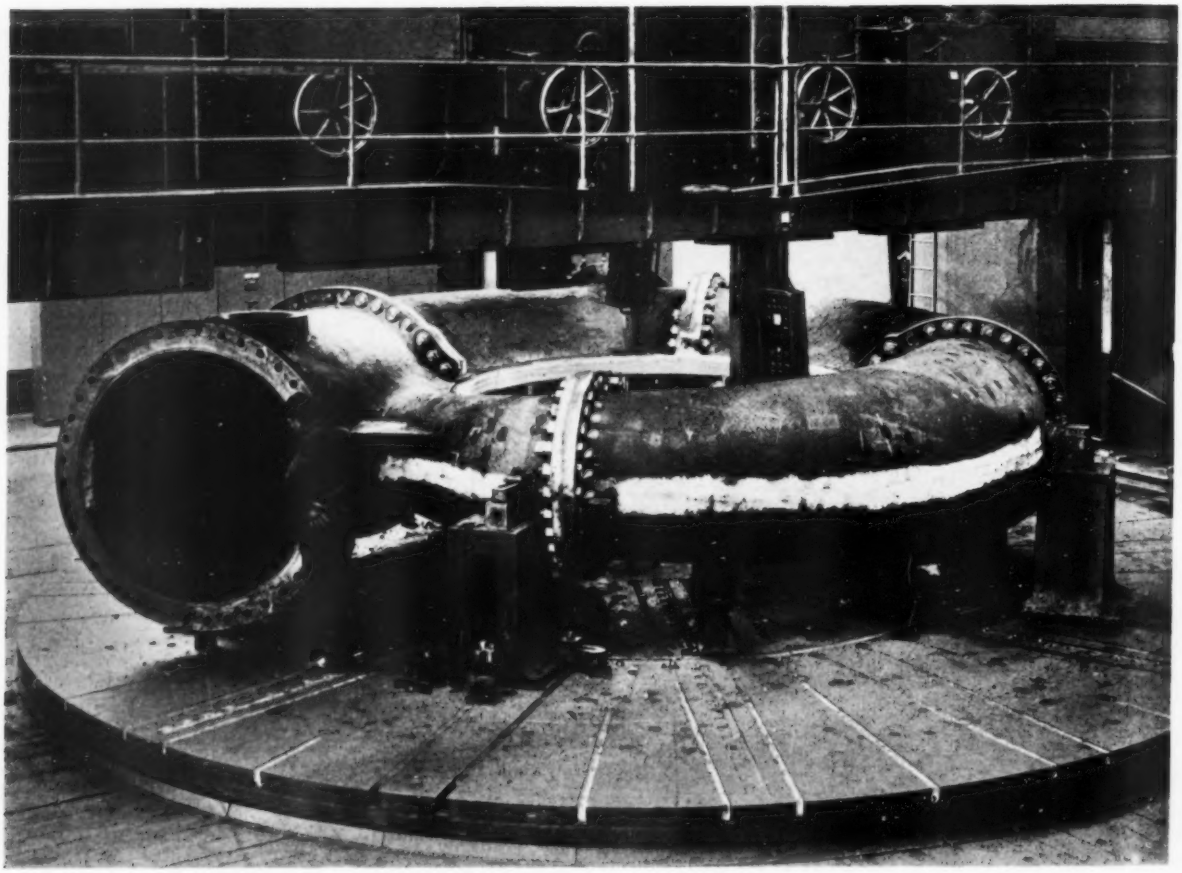



TECHNOEXPORT

FRANCO TOSI S.p.a ★ LEGNANO (ITALY)

HYDRAULIC TURBINES

KAPLAN ★ FRANCIS ★ PELTON ★ GOVERNORS ★ VALVES



Spiral Casing for Francis turbines, being machined on the lathe

AZIENDA COMUNALE ELETTRICITA' ED ACQUE - ROMA - (ITALY)

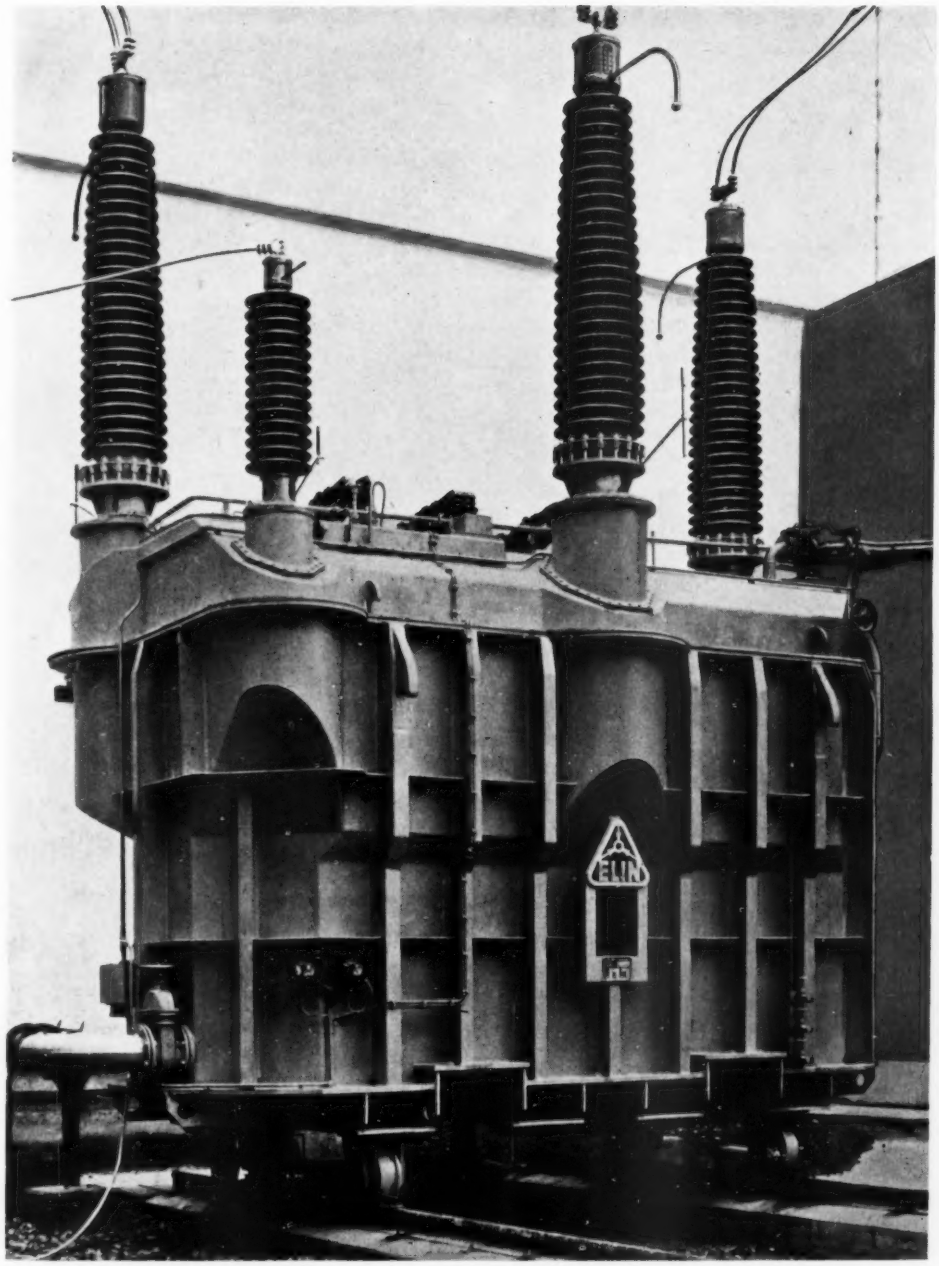
S. ANGLO HYDRO-ELECTRIC POWER STATION

Two Francis turbines, each designed for:

Net head: 156.50 mts. Output: 29,830. kW

Discharge: 21,800. = lts/sec. Rotative speed: 375. = r.p.m.

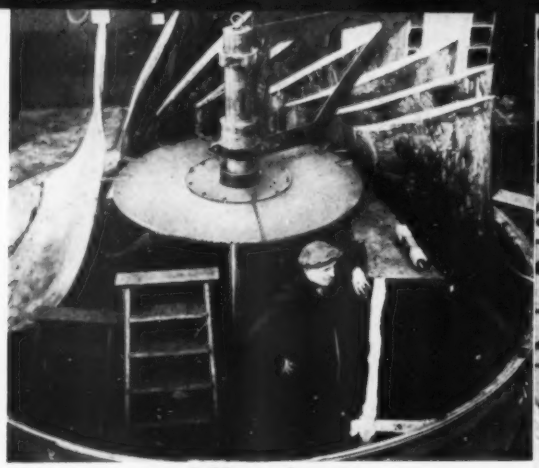
POWER TRANSFORMERS



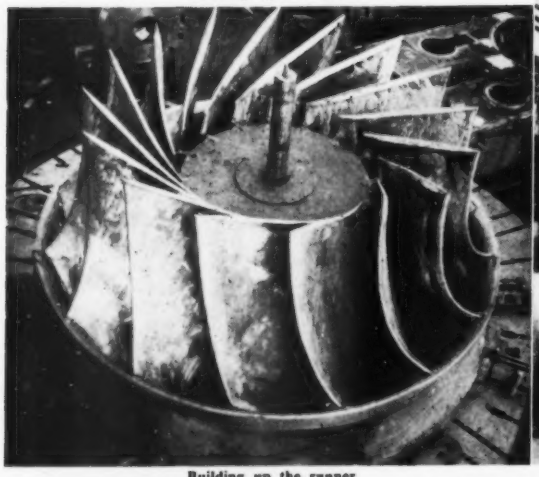
One of three unit transformers supplied to Lünersee Power Station of Vorarlberger Illwerke Aktiengesellschaft; 56 MVA, 10.5 kV/240 kV, with brought-out HV-neutral, with external water coolers and heat exchangers, the latter for a heating installation using lost heat.

"ELIN"

AKTIENGESELLSCHAFT FÜR
ELEKTRISCHE INDUSTRIE
VIENNA - AUSTRIA



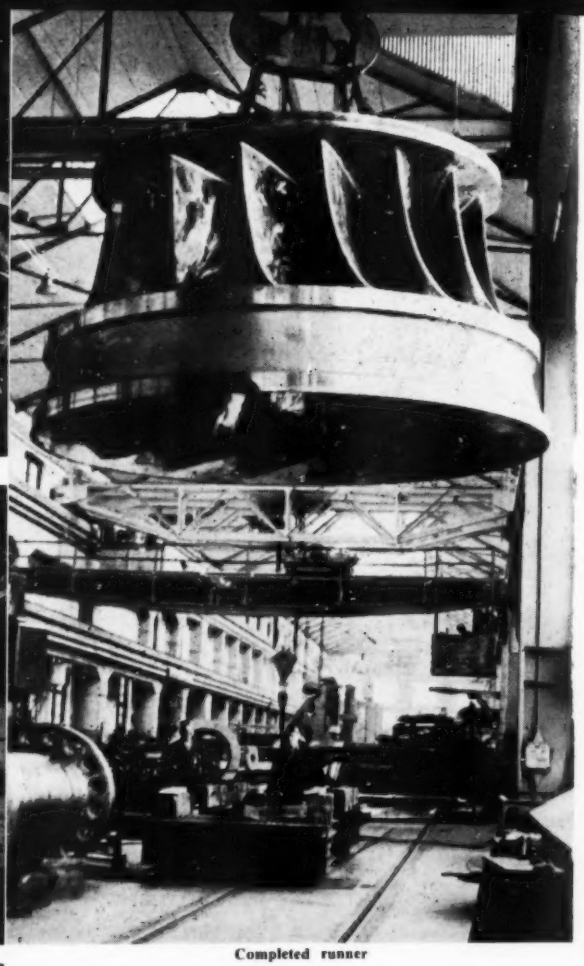
Building up the runner



Building up the runner

Welding the runner

Stress relieving furnace



Completed runner

The third **KMW** Turbine for the Swedish power plant Harrsele has now been put into service

With the installation of the third KMW turbine in the Harrsele Power Station, this plant has now been completed.

The first unit was put into service on August 15th, 1957, the second on January 29th, 1958, and the third on April 22nd, 1958. The total output amounts to 264,000 HP at a net head of 52.6 m (173 feet) and 125 r.p.m.

The runners are of welded construction according to our patented method. This construction ensures a better quality product and an improved hydraulic efficiency.

AB KARLSTADS MEKANISKA WERKSTAD

Karlstad - Sweden

Representatives in Canada: A. Johnson & Co. (Canada) Ltd.
607 Shell Tower Building, Montreal 2

Reinforced Concrete Pipes, ordinary and pre-stressed in all diameters and pressures

Condotta in cemento armato precompresso - diametro m.3.30 - pressione 9 atmosf.
Impianto idroelettrico di Brunico (Italia) - Soc, Montecatini

SACAIM

Soc. p. Az. Cementi Armati Ing. Mantelli

VENICE (ITALY) - Accademia 979/A - Telephone 24.072

ESCHER WYSS

High-pressure Francis runner for Fionnay Power Station (Switzerland)

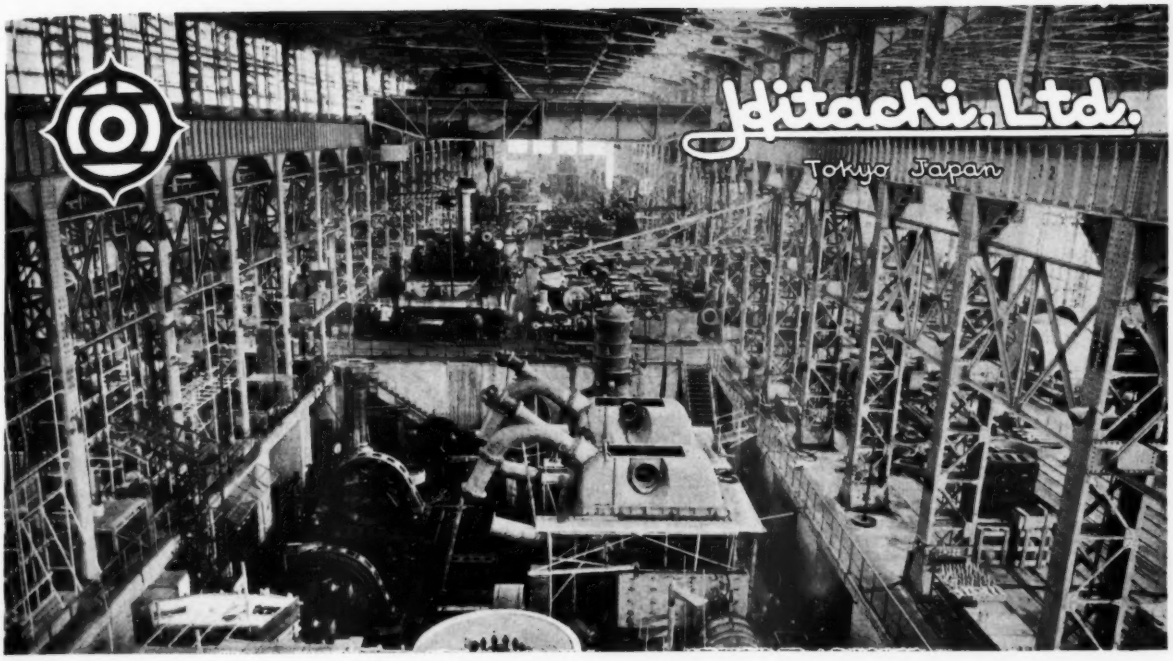
High-pressure Francis turbines in Switzerland

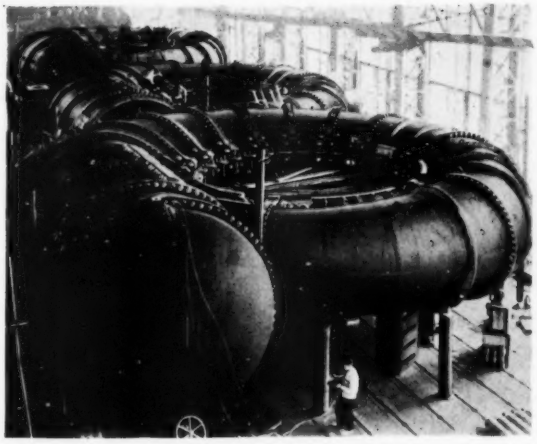
In operation:	Safienplatz	Fionnay
Max. head	422 m.	455 m.
Output	60000 HP.	63000 HP.
Speed	750 r.p.m.	750 r.p.m.

In manufacture:	Sils	Ferrara*
Max. head	412 m.	520 m
Output	88000 HP.	100000 HP.
Speed	600 r.p.m.	750 r.p.m.

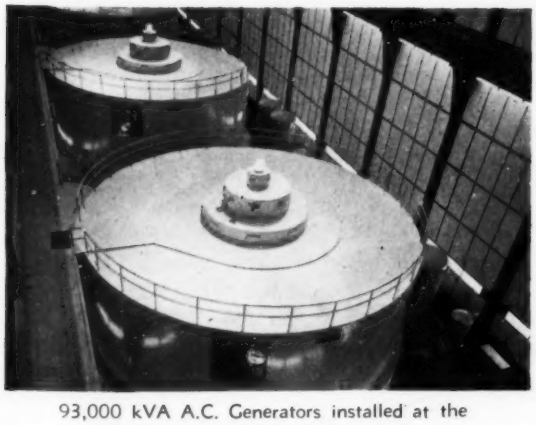
Escher Wyss Ltd., Zurich/Switzerland

* In conjunction with Charmilles S.A.





Casings for 150,000 HP Francis Turbines for the Bahkra Power Plant, India.



93,000 kVA A.C. Generators installed at the Sakuma Power Plant, Central Japan.

Dependable Electrical Equipment for World Industry

Shown above is a section of one of Hitachi's vast, modern plants where superior-quality equipment, including turbines and generators, is produced. Hitachi products are widely known in other countries and have been exported to India, Brazil, Argentine, Mexico and other destinations.

- WATER TURBINES • STEAM TURBINES • BOILERS
- A.C. GENERATORS • D.C. GENERATORS
- FREQUENCY CHANGERS • ROTARY CONVERTERS
- MERCURY RECTIFIERS • POWER TRANSFORMERS
- INDUCTION REGULATORS • STATIC CONDENSERS
- CIRCUIT BREAKERS • DISCONNECTING SWITCHES
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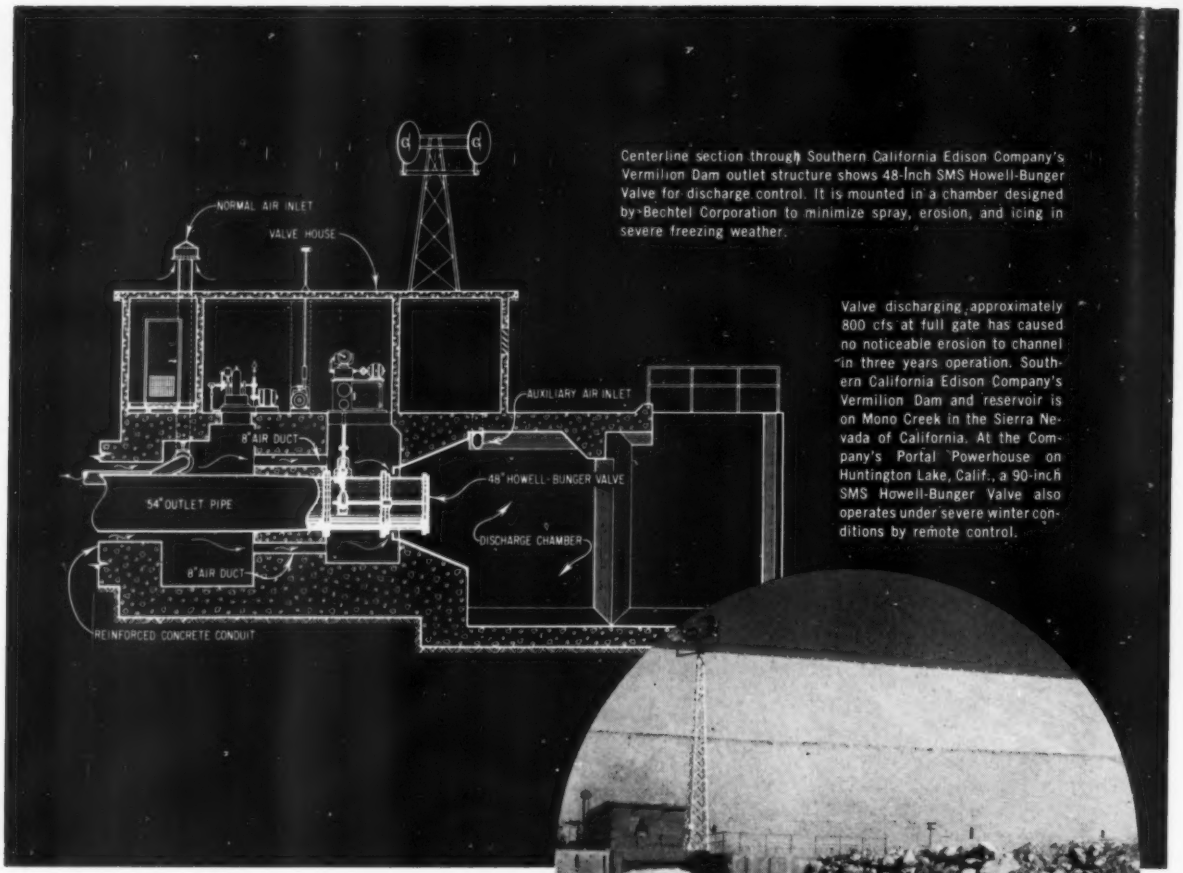
Cable Address: "HITACHY" TOKYO



Centerline section through Southern California Edison Company's Vermilion Dam outlet structure shows 48-Inch SMS Howell-Bunger Valve for discharge control. It is mounted in a chamber designed by Bechtel Corporation to minimize spray, erosion, and icing in severe freezing weather.

Valve discharging approximately 800 cfs at full gate has caused no noticeable erosion to channel in three years operation. Southern California Edison Company's Vermilion Dam and reservoir is on Mono Creek in the Sierra Nevada of California. At the Company's Portal Powerhouse on Huntington Lake, Calif., a 90-inch SMS Howell-Bunger Valve also operates under severe winter conditions by remote control.

Remote Controlled SMS Howell-Bunger Valve at Vermilion Dam Provides ***EASY REGULATION OF FREE DISCHARGE***

A 48-inch SMS Howell-Bunger Valve was chosen for Vermilion Dam to provide close, accurate regulation of releases from 10 cfs to 700 cfs. Remote control was necessary because of the isolated location of the site, and dependable operation and minimum maintenance were required, even in 20° below zero winter weather. Since late in 1954, this Howell-Bunger Valve has operated successfully under all conditions.

Rugged, simply designed and low in initial cost, the SMS Howell-Bunger Valve gives you:

- Free discharge with safe energy dissipation.
- Hydraulic balance without vibration.
- High coefficient of discharge for an energy dissipating valve.
- Minimum cavitation and maintenance.
- Simplicity of operation.
- Only one moving part in contact with stream.
- Discharge into atmosphere or under submerged conditions.

To obtain complete information, write to S. Morgan Smith Company, York, Pennsylvania.

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enables generators and lines to be connected to the system rapidly and reliably even when the system frequency and voltage deviate widely from their nominal value as a result of faults. Moreover, it saves precious time and renders valuable service in both power stations and substations.
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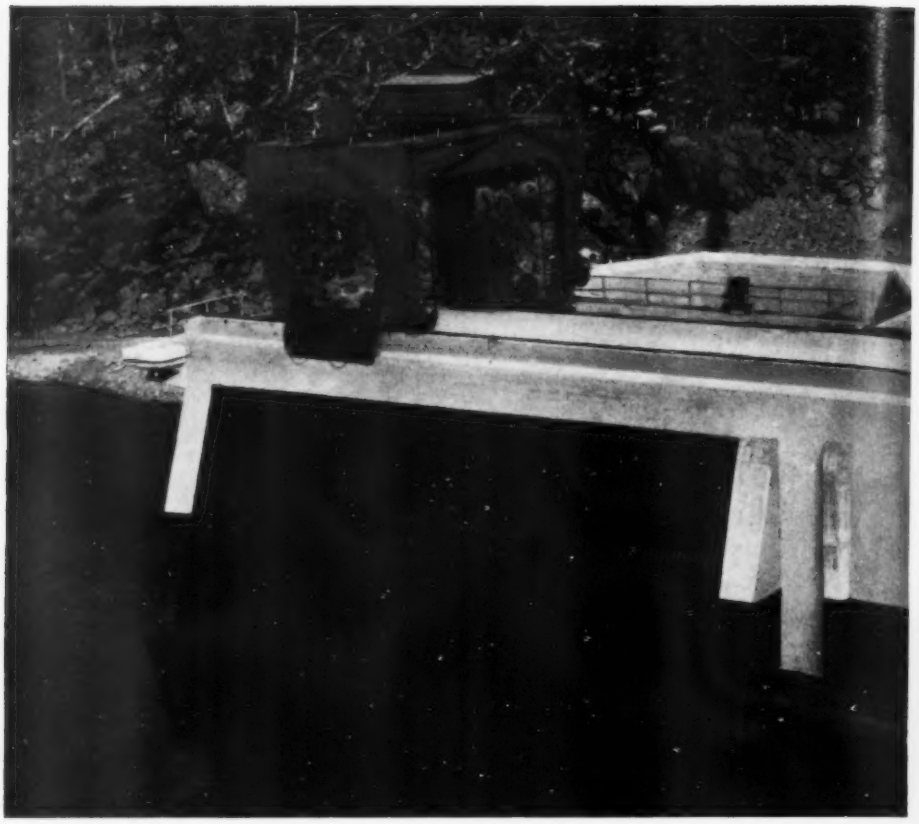
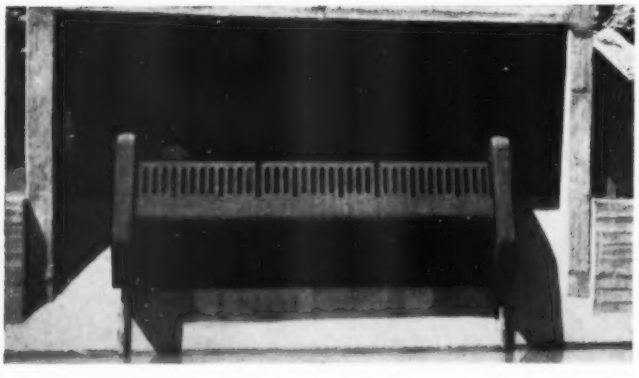


At the intakes . . . .

SCREENS AND CLEANING GEAR



Screens, generally of the built-up bar type, can be supplied in fixed or removable styles for tunnel or turbine intakes, to suit requirements.

For cleaning fixed bar screens, an efficient means is offered by the Glenfield Plough Rake. Two patterns of the Rake are available, as illustrated: to span a screen partially, with traversing operating gear; and to span a screen fully, with fixed operating mechanism.

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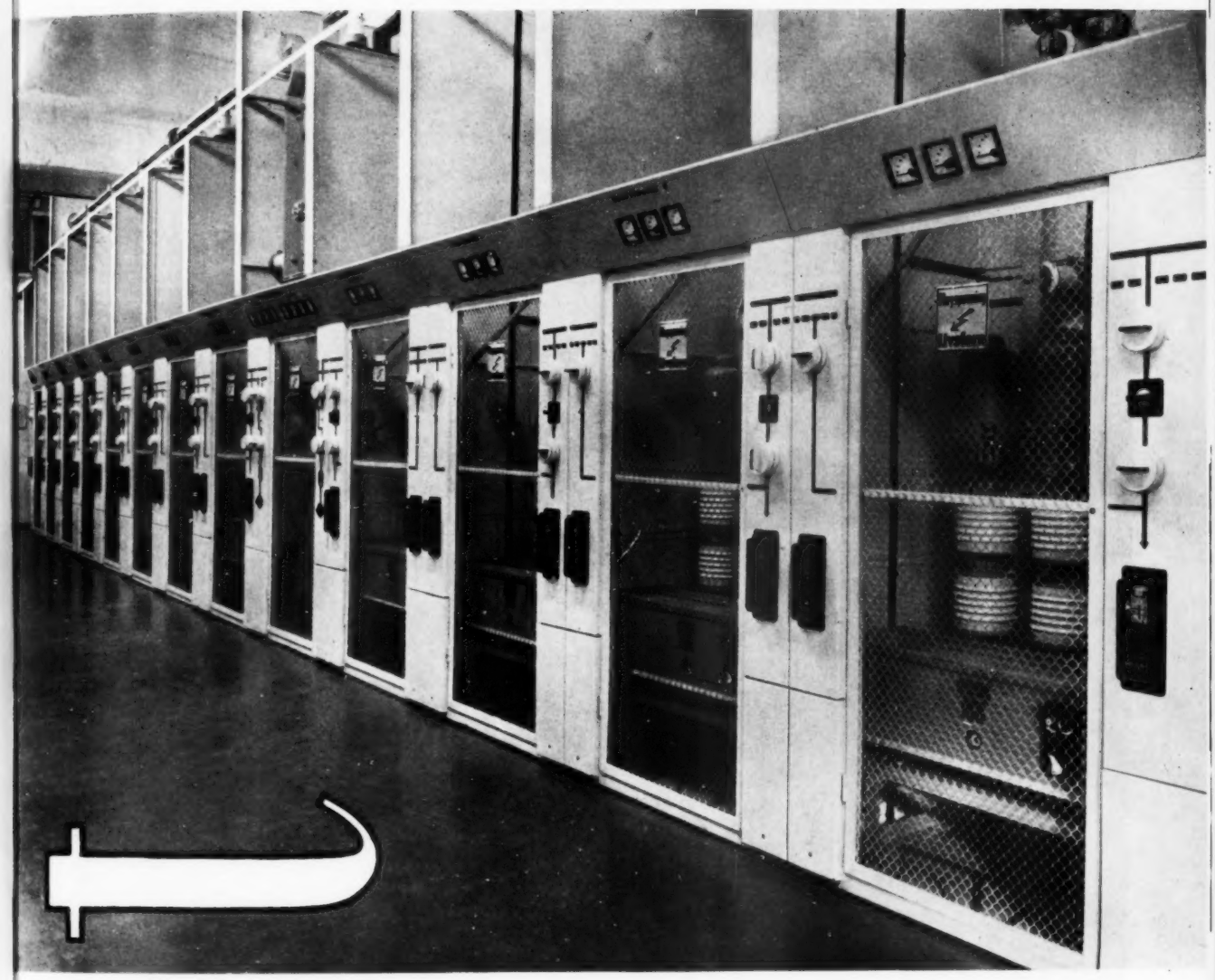
circuit breakers, mt type



mt

Scarpa & Magnano

SAVONA-ITALY

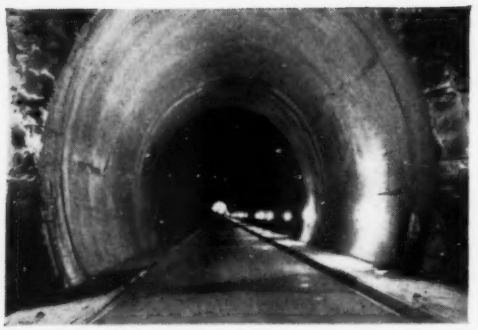
The above shown circuit breakers, MT/150-P model for 10 kV rated voltage, are part of a supply for Messrs. PETERSON & SON, MOSS (Norway).

This small oil circuit breakers series for medium voltage—from 10 to 45 kV—successfully underwent all type tests, as stated by I.E.C. Standards, performed at the KEMA Laboratory (Holland), conquering in a couple of years the international market.

Hundreds of MT circuit breakers have been in fact supplied, not only in Italy, but also to Finland, Sweden, New Zealand, Ireland and Turkey, everywhere obtaining a bright success.

40 miles of tunnelling

New high speed tunnelling techniques have been employed with outstanding success on contracts executed by The Mitchell Construction Company for the North of Scotland Hydro-Electric Board. The illustration shows the junction of the Doe and Ceannacroc Tunnels on the Glenmoriston Scheme, whilst the inset shows the access tunnel to Ceannacroc Underground Generating Station. Included in the Company's contracts with the Board is nearly 40 miles of hard rock tunnelling, of which 27 miles has already been driven.



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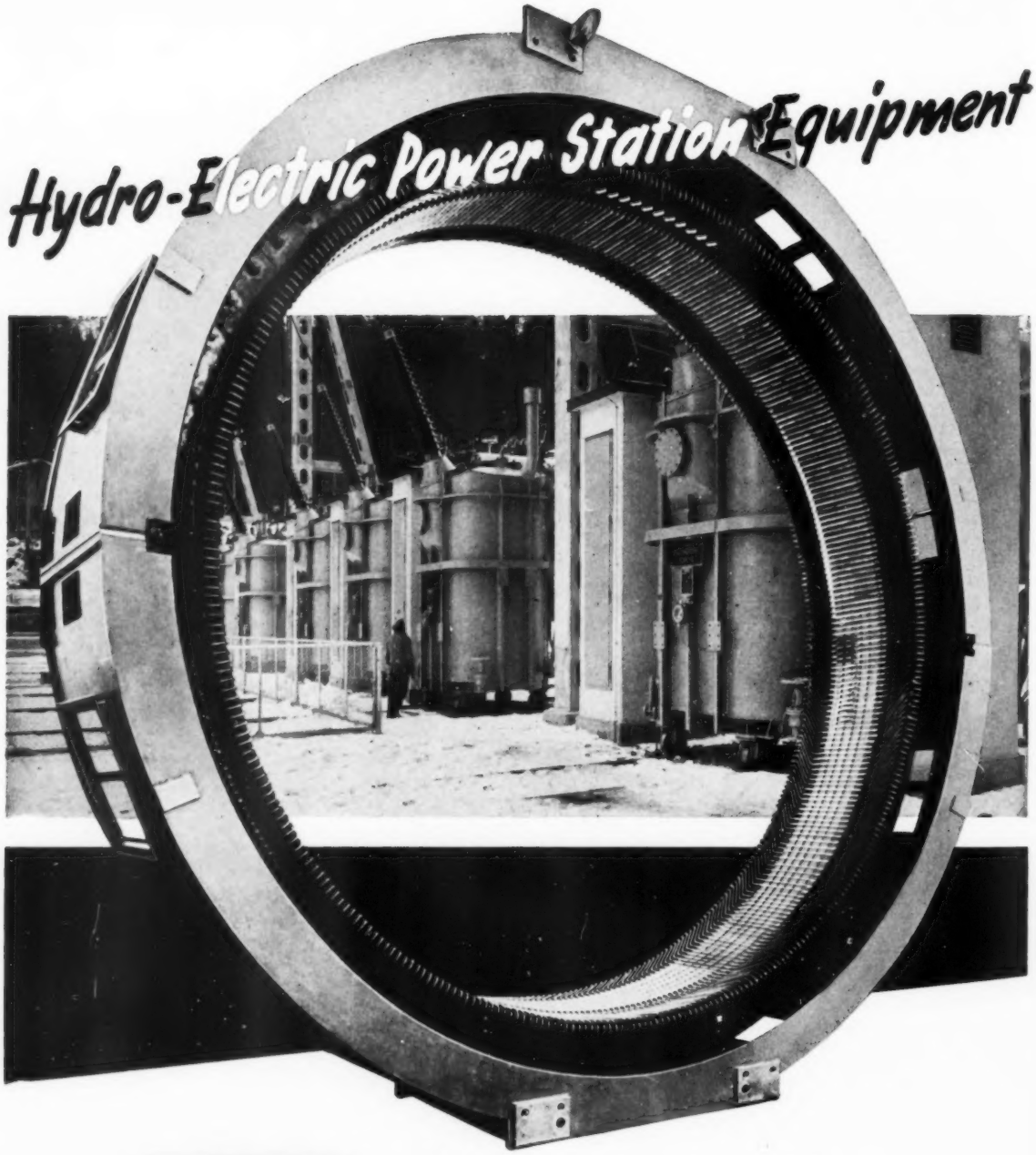

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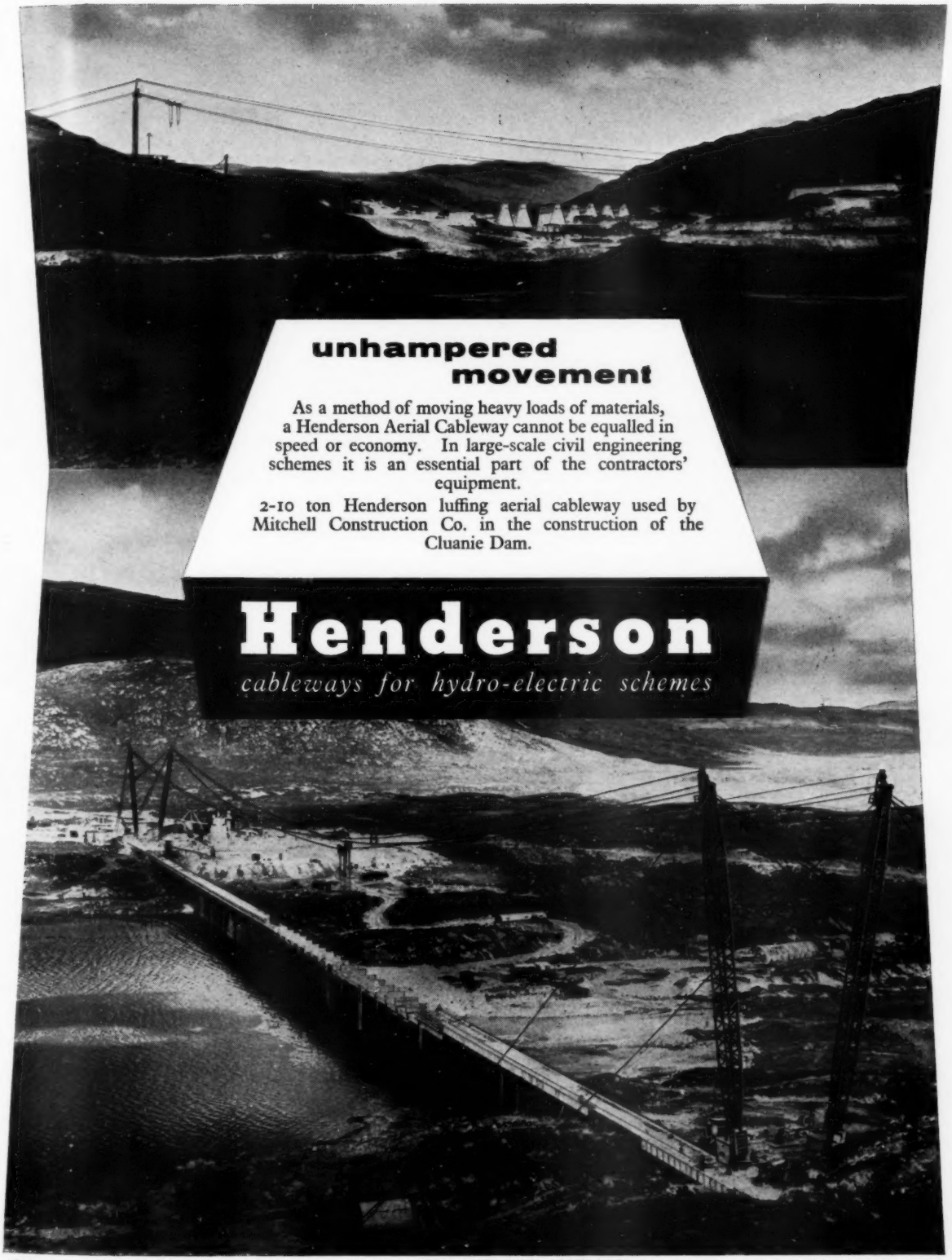
unhampered
movement
As a method of moving heavy loads of materials, a Henderson Aerial Cableway cannot be equalled in speed or economy. In large-scale civil engineering schemes it is an essential part of the contractors' equipment.
2-10 ton Henderson luffing aerial cableway used by Mitchell Construction Co. in the construction of the Cluanie Dam.
Henderson
cableways for hydro-electric schemes

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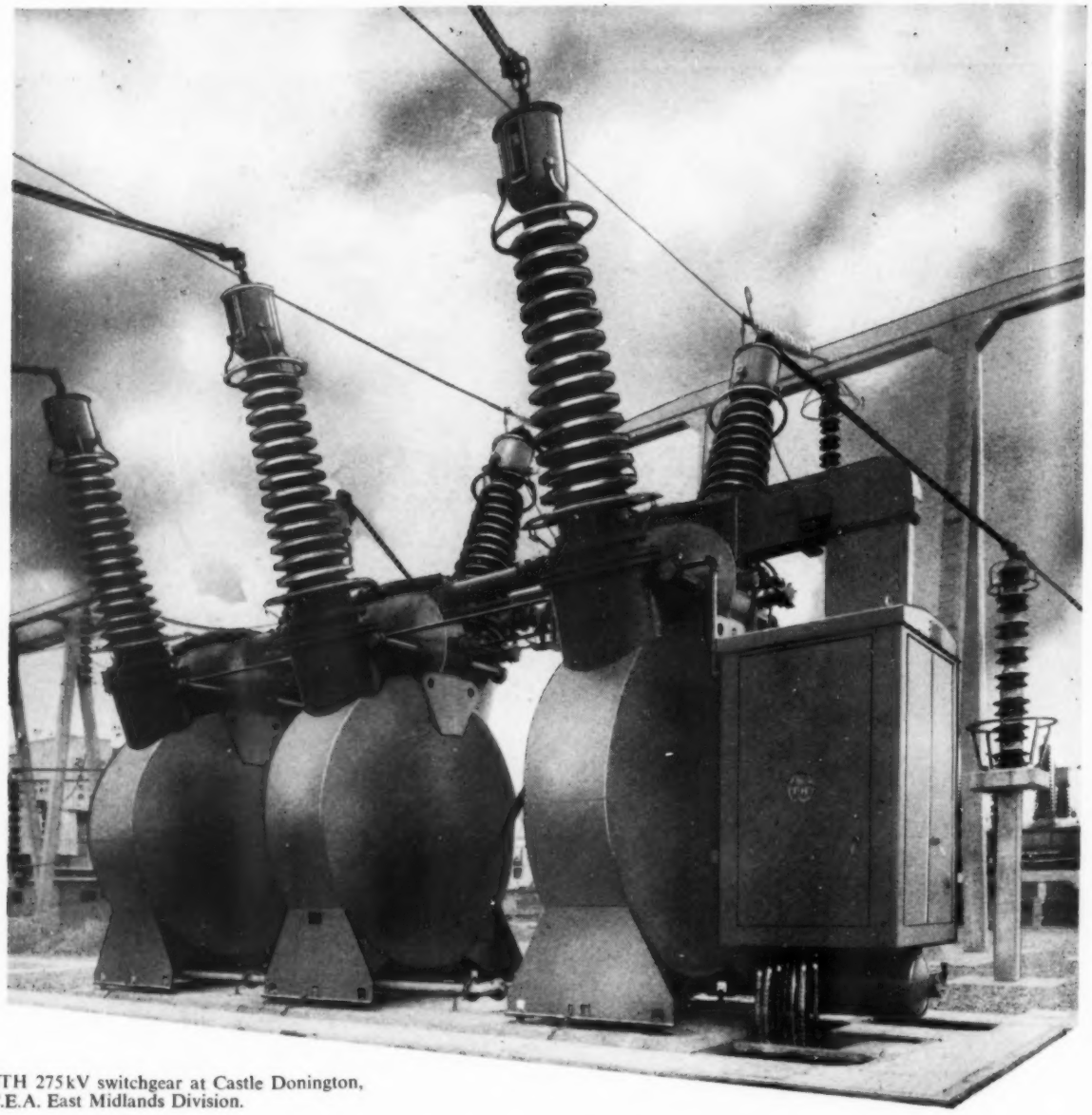

BTH 275kV switchgear at Castle Donington,
C.E.A. East Midlands Division.

B
T-H

330-kV SWITCHGEAR
for
KARIBA

The two million pound order awarded to British Thomson-Houston for the Kariba Hydro-electric Scheme includes 23 oil circuit-breakers of the BTH lenticular type. These units are a direct development of those installed on the 275kV British Super-Grid.

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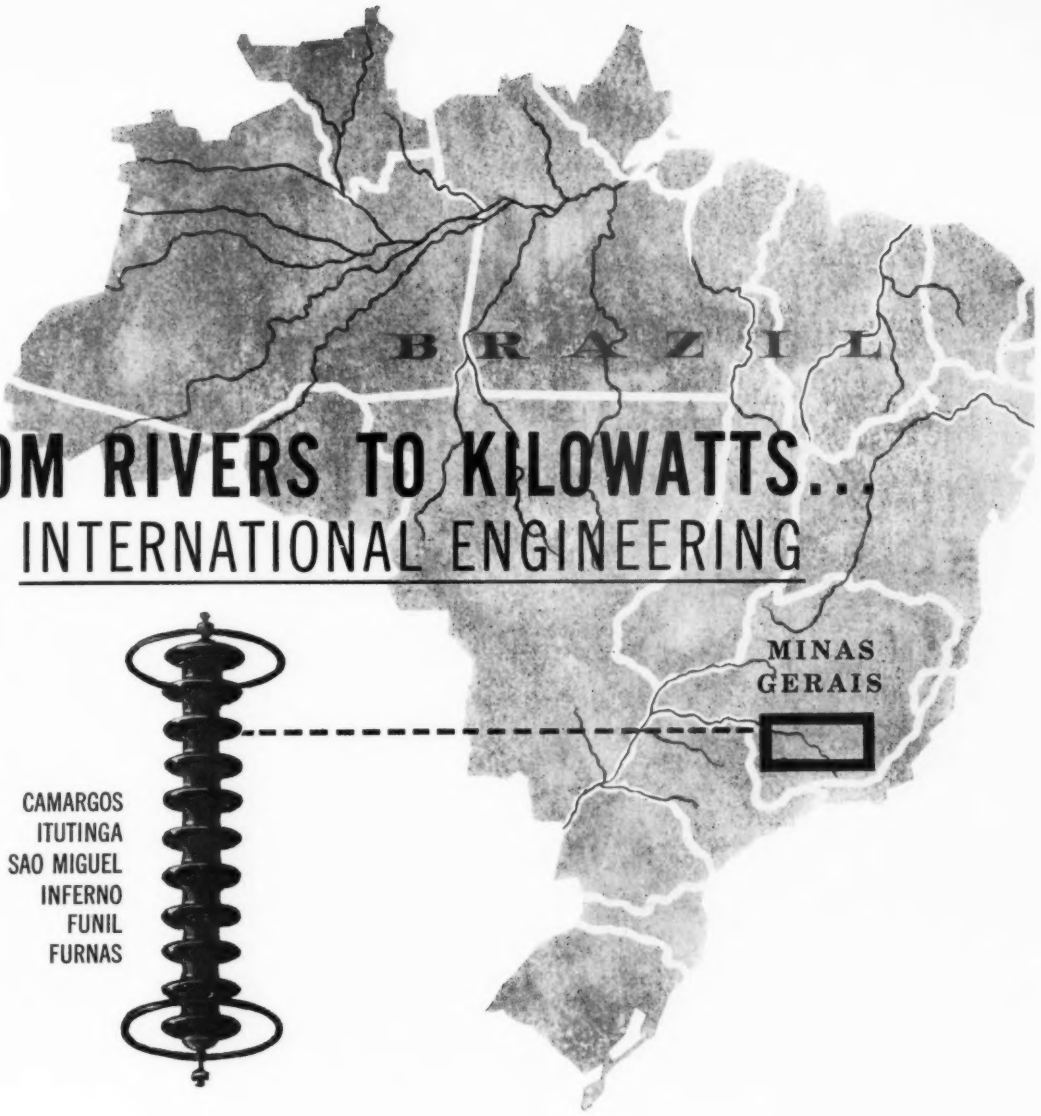

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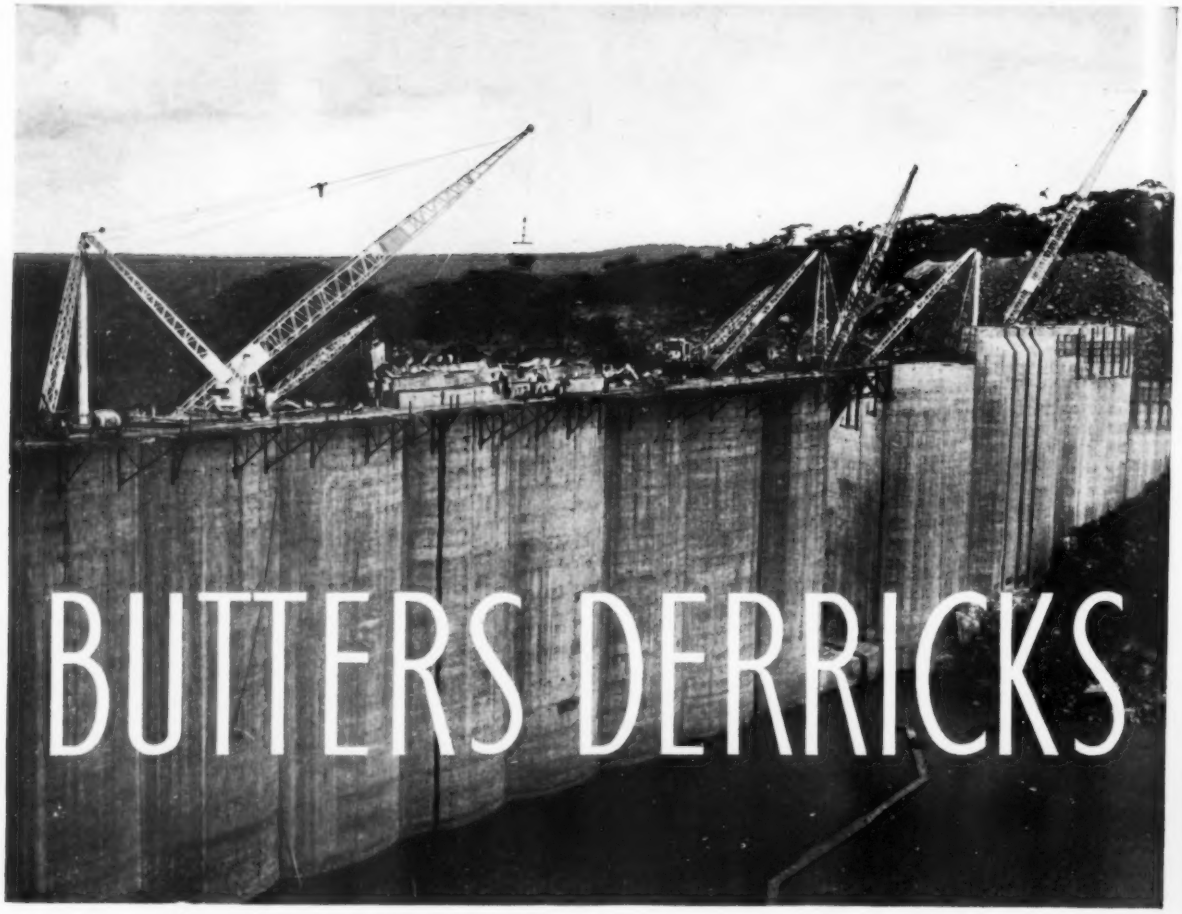
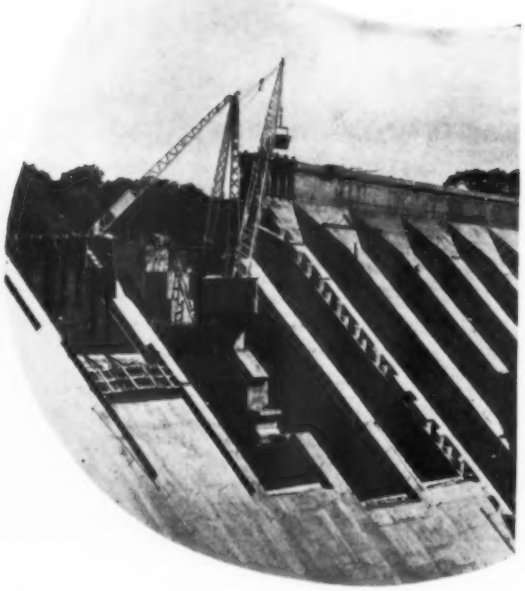


The Dam Builders



These pictures show several 7 ton Butters Derrick cranes in use on the construction of a dam in Rhodesia.

The difficult conditions and terrain often encountered in such work demand a very high degree of efficiency in the plant and ability to keep going with minimum maintenance. Butters Derricks have proved ideal—on dam construction all over the world, from New Zealand to the North of Scotland, from Iraq to Eire. Electric, Steam or Diesel drive can be provided in sizes to suit every purpose.

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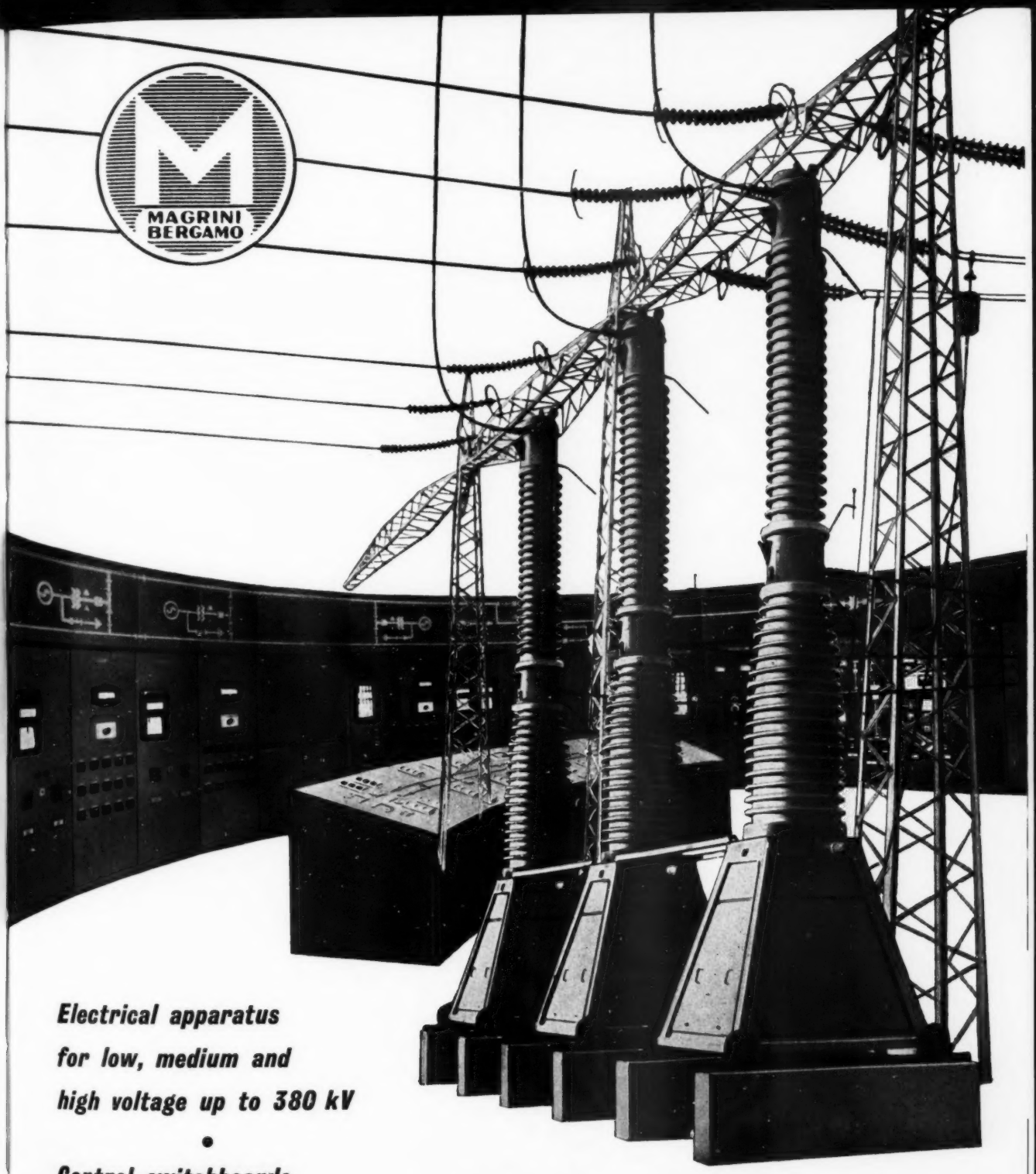

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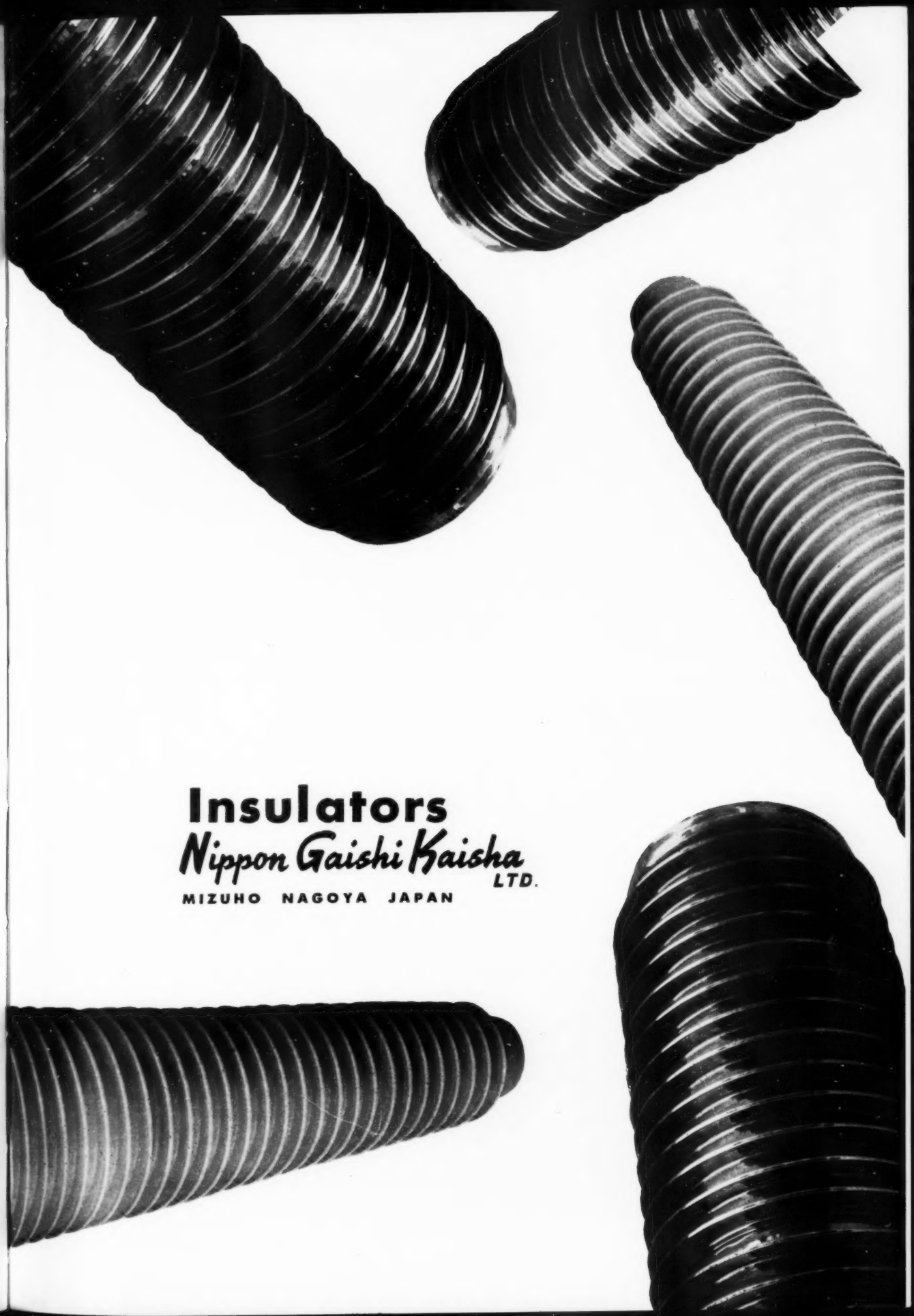
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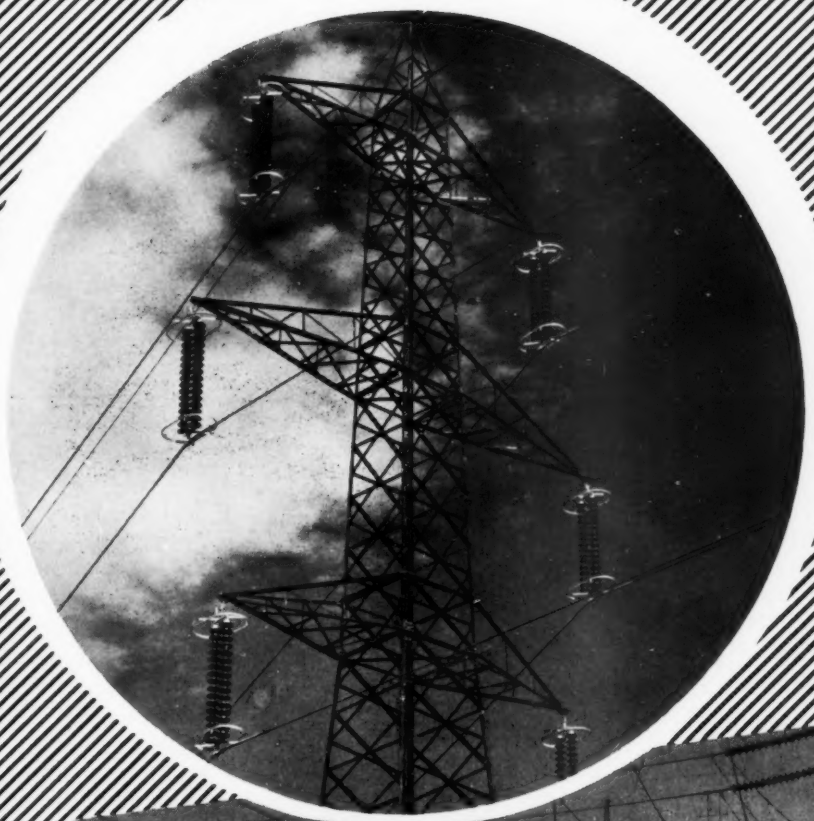

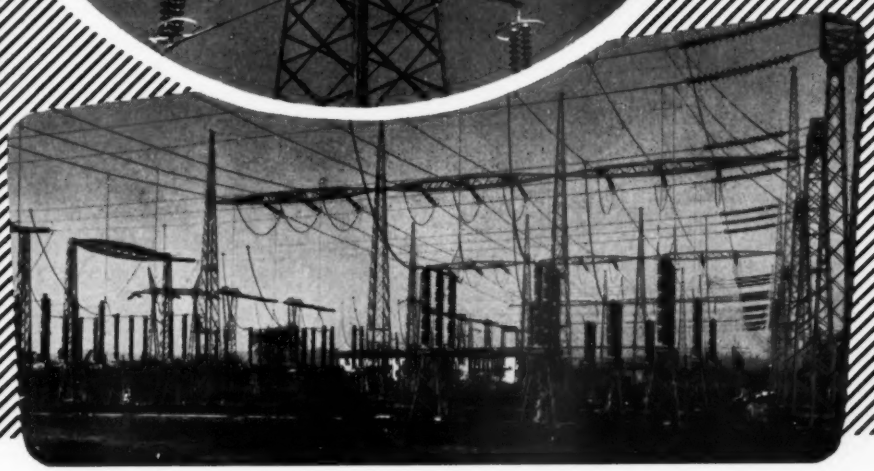

For 30 years Messrs. A. Salvi and Company, Milano, Italy, have specialized in the design and manufacture of complete equipment for Overhead Transmission Lines, including Compression Dead End Assemblies and Joints.
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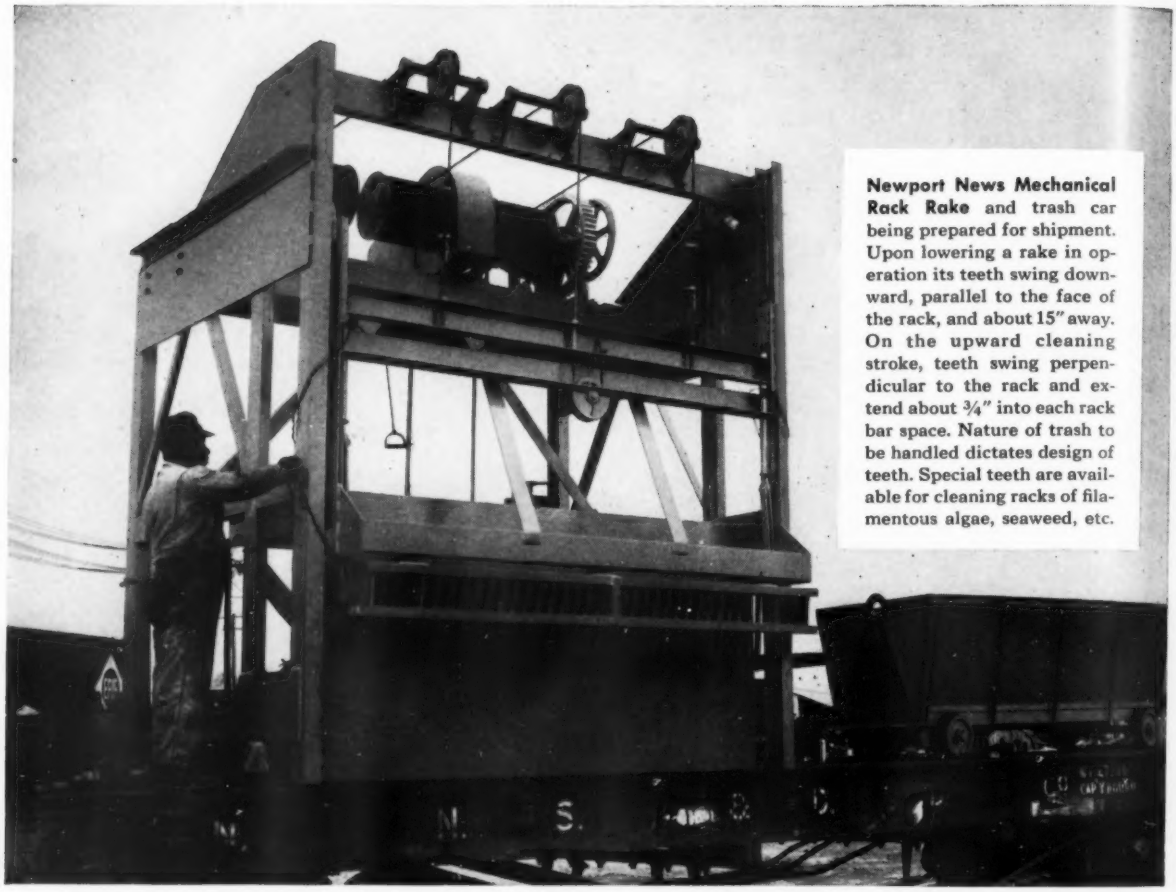

Newport News Mechanical Rack Rake and trash car being prepared for shipment. Upon lowering a rake in operation its teeth swing downward, parallel to the face of the rack, and about 15" away. On the upward cleaning stroke, teeth swing perpendicular to the rack and extend about ¾" into each rack bar space. Nature of trash to be handled dictates design of teeth. Special teeth are available for cleaning racks of filamentous algae, seaweed, etc.

Positive cleaning action calls for guided rack rakes

Excessive loads of trash cannot push a Newport News rake away from the racks.

And there's no chance of the rake *riding over trash* on the upward cleaning stroke. Nor will its teeth drop the debris. Because Newport News builds the rake to operate in channel guides... for *positive cleaning action.*

That's why efficient rack cleaning assures the maximum head at intakes and is yours when you put a rugged and dependable Newport News Mechanical Rack Rake on the job.

Maximum width of a single rake is determined by design of the forebay. Where a single bay is extremely wide, intermediate guides are sometimes installed to reduce width of the rake. Such installation is relatively inexpensive.

A power-operated Newport News Mechanical Rack Rake, under ordinary conditions, enables one man per shift to keep racks clean for a dozen bays, from 5'-6" to 28'-6" in width.

Trash is cleaned from the rake manually, or by a mechanical "sweep", into a flume, a car or onto a trash apron. Local conditions and nature of the trash dictate method of disposal. The many typical installation arrangements fit most needs, but each installation is a custom built job.

We invite inquiries from water users who are troubled with trash. Write for your copy of "RACK RAKE", an illustrated booklet describing the operation and advantages of the Newport News Mechanical Rack Rake.

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Shipbuilding and Dry Dock Company
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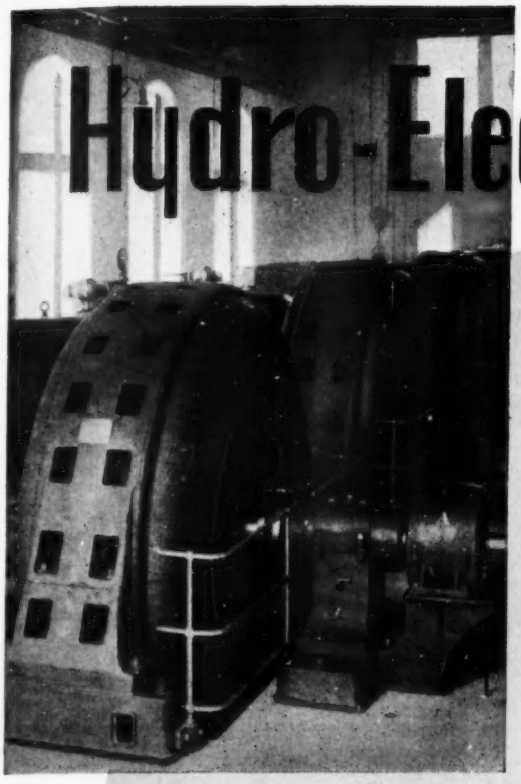
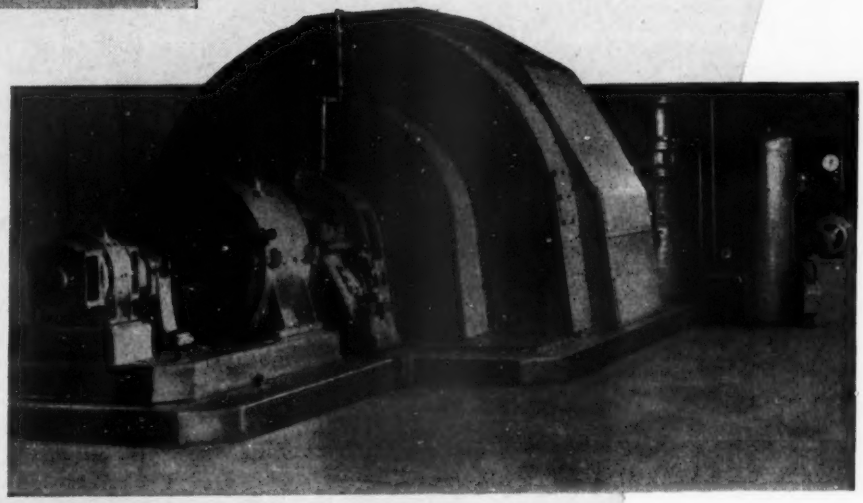


Hydro-Electric Generators

Fifty years ago Bruce Peebles engineered the Snowdon Hydro-Electric Scheme — from tunnelling to transmission lines—the first large project for public supply in Great Britain.

The original Cym Dyli Power Station, pictured at left, is equipped with four 1,500 kVA twin pelton wheel driven generators, which are still running.

These early generators were the fore-runners of the large machines being built to-day for service in many parts of the world.



14,000 kVA hydro-electric generator installed in the Yksnelvane power station, Firdakraft, Norway.

Half a century has elapsed since the first Hydro-electric scheme was completed. The pioneering spirit is still dominant and is revealed to-day in the many contracts undertaken for large hydro-electric plant for many schemes at home and abroad. Ample resources for manufacture and testing enable the largest contracts to be undertaken.

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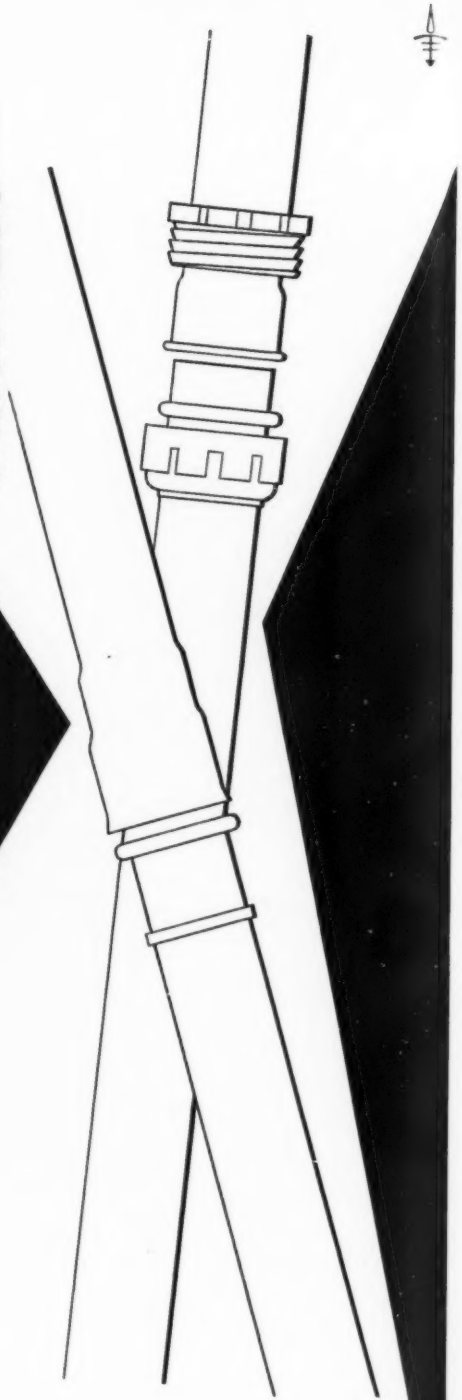

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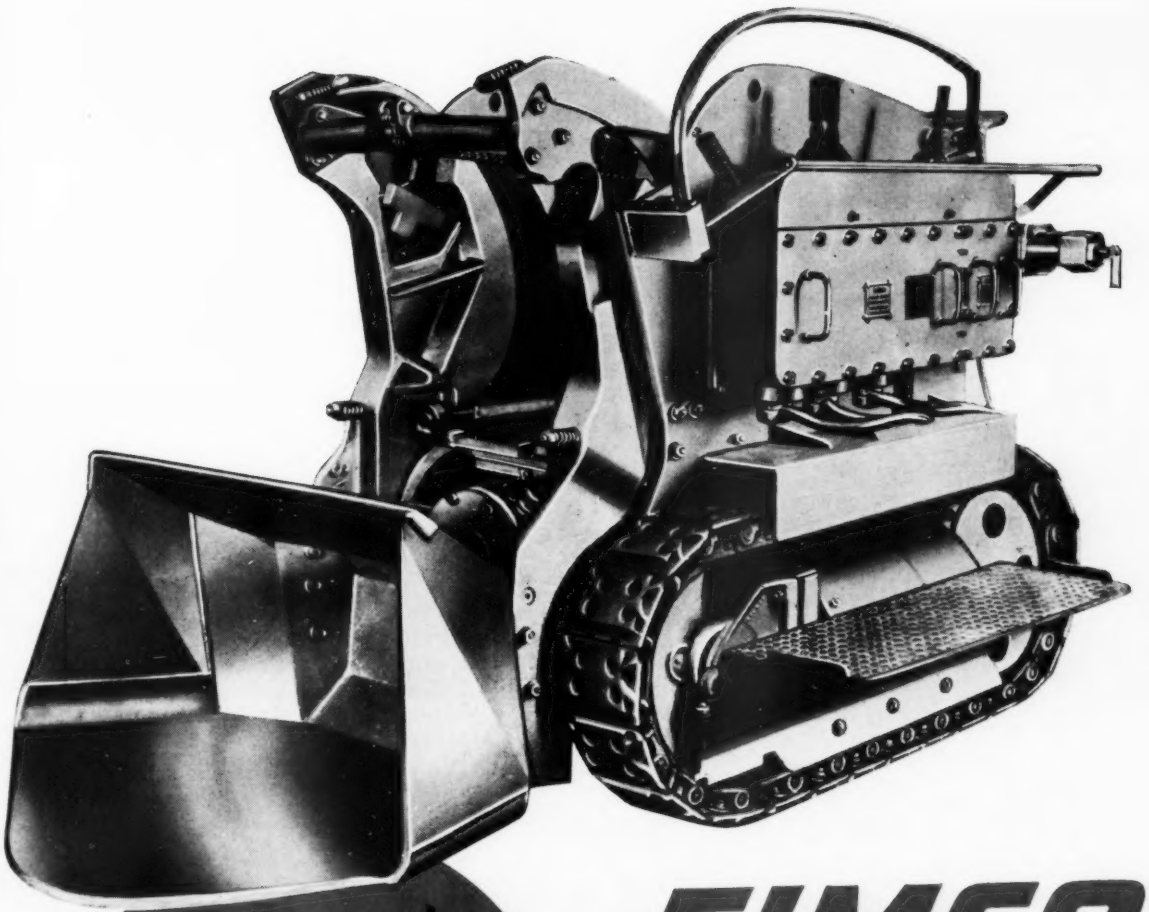

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P.1718

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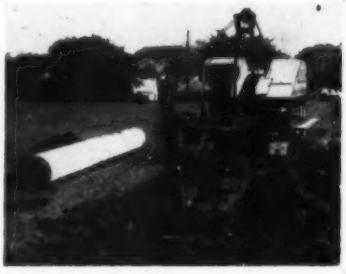

19-RB ¾ cu. yd. dragshovel excavating pipe trenches for a water supply

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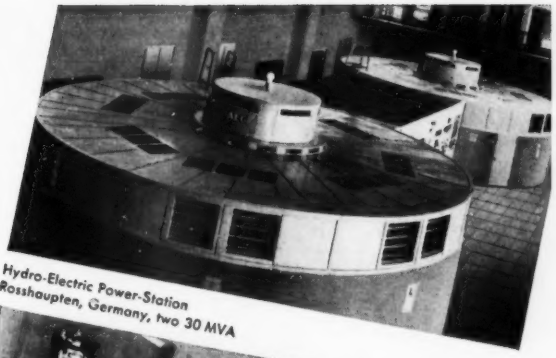
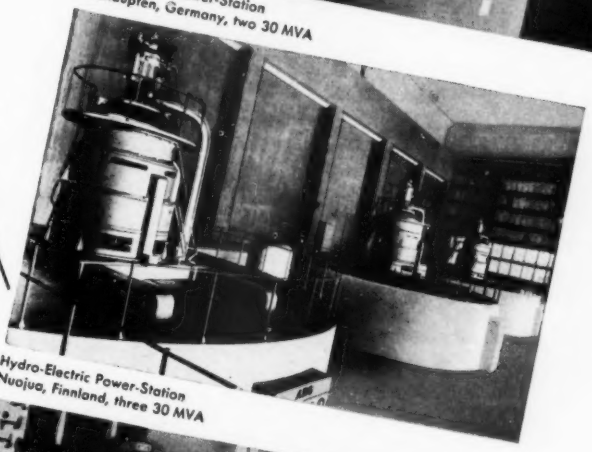
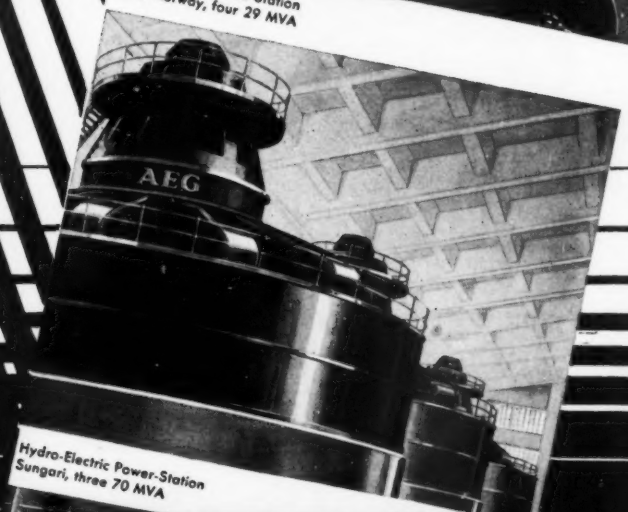


Hydro-Electric Power-Station
Rosshaupten, Germany, two 30 MVA



Hydro-Electric Power-Station
Nuojua, Finnland, three 30 MVA

Hydro-Electric Power-Station
Nore, Norway, four 29 MVA



Hydro-Electric Power-Station
Sungari, three 70 MVA

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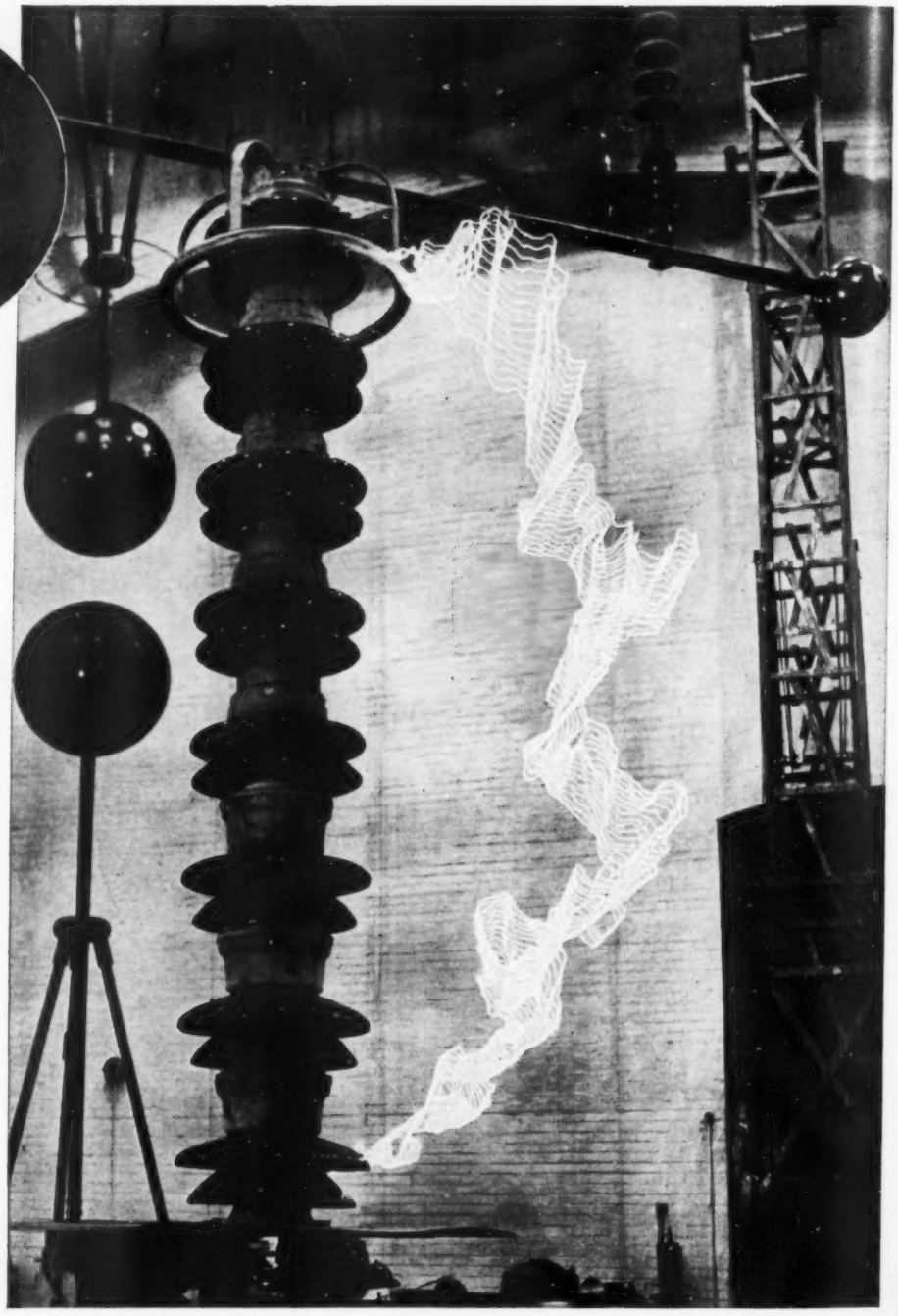
275kV. post insulators for the British Super Grid



One of the S.P.P. 8-unit post insulators for the B.E.A. 275 kV Super Grid undergoing the 50-cycle flashover test at 705 kV during the official type tests carried out at the million-volt laboratory at Stourport-on-Severn.

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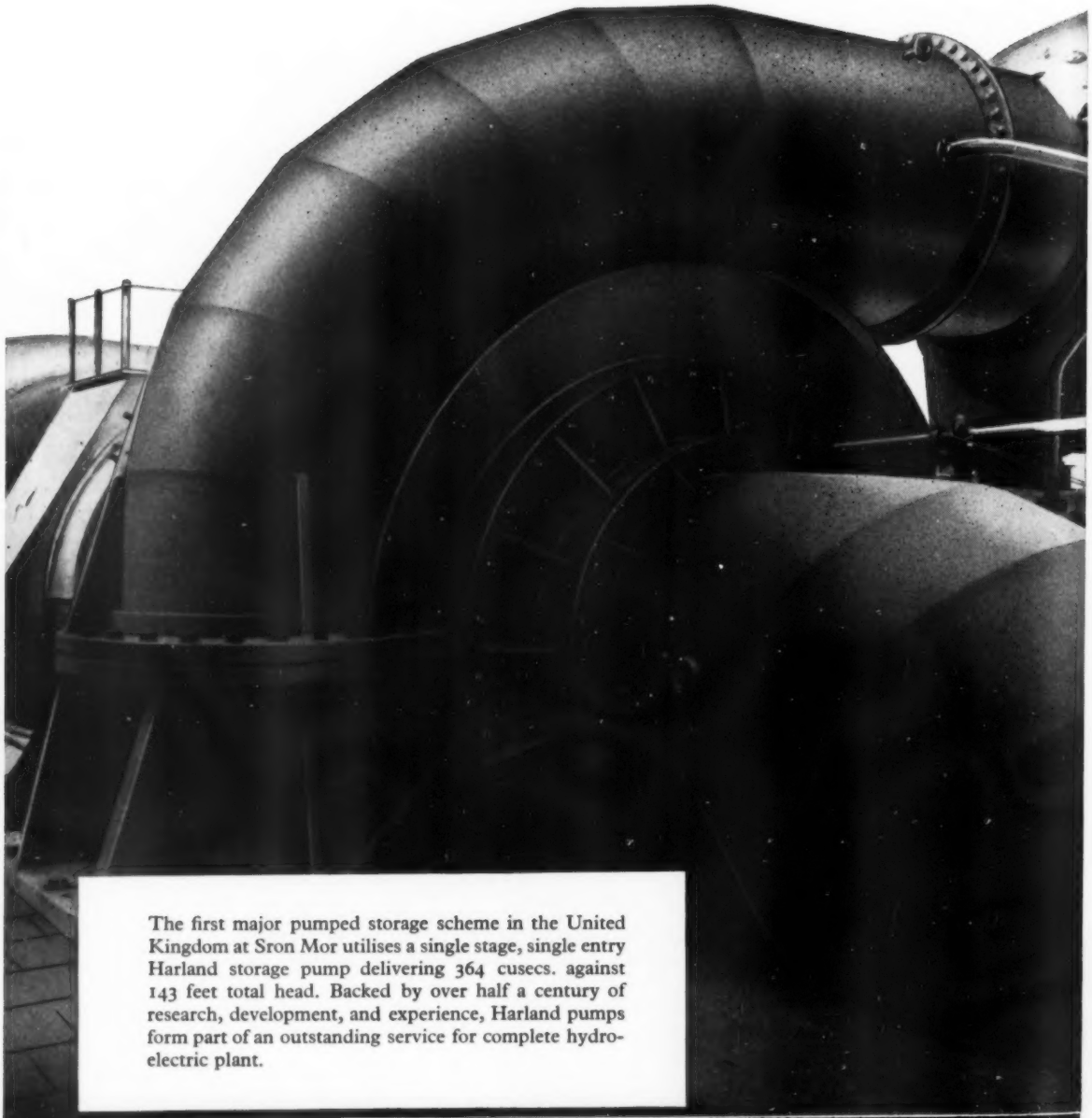
storage pump at Sron Mor



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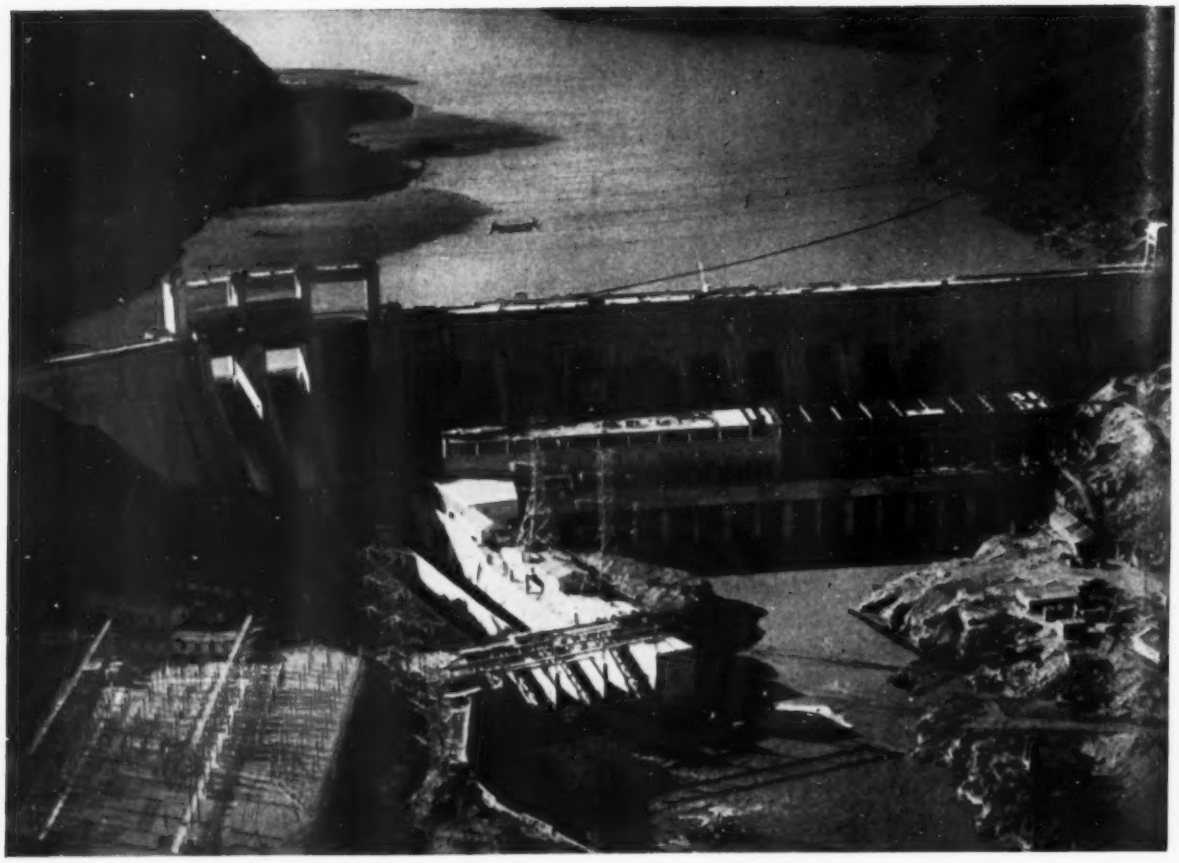

New power for a Dominion from the Roxburgh Hydro-Electric Project

New Zealand's largest hydro-electrical undertaking, the Roxburgh project, will eventually feed an additional 320,000 kilowatts of electricity into the South Island grid system. The 1,200 ft. Dam, which is the largest ever built in the Dominion, was completed far ahead of schedule by Cubitts' New Zealand Co., in association with Conrad Zschokke Ltd., Geneva, and Downer & Co. Ltd., New Zealand.

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WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

Power Development in Austria

SINCE the war a number of important hydro-electric installations have been built in Austria, and from time to time we have published articles describing some of them, but a tour of Austrian hydro-electric developments which we were recently privileged to make as the guest of the Österreichische Elektrizitätswirtschafts-A.G. left us with a new and much keener appreciation of the magnificent recovery in power supply which Austria has made since the war. After 1945—to describe the situation in the manner in which it was related to us—Austria was left with "a few ruins." Quite apart from the war damage sustained, the Austrian power system had been geared to German military needs, and work had been commenced on a number of schemes—notably Glockner-Kaprun and Ybbs Persenbeug — but had been abandoned.

Thus at this period Austria was faced with a severe shortage of power-generating capacity and of the labour, materials and finance to make good the deficiency. An index of the position as it then obtained, and of the transformation that has since taken place, is available from the figures of electricity consumption. In 1946 the total consumption was 3,059 GWh; in 1956 it had risen to 10,504 GWh. These figures, of course, reveal an advance far in excess of the "doubling in ten years" rule that serves as a rough guide in assessing the increase in demand. Installed capacity (both hydro and thermal) in 1947 was 1,600 MW, and by the end of 1957 it had increased to 3,200 MW.

It was realised at the outset that Austria's economic recovery would depend on the provision of an adequate power system and that the pattern of her resources favoured generation primarily from water power. The first vital step was taken in 1947, when a Federal Law was passed to nationalise the supply systems of the country. Although some 1,200 undertakings were taken over, this was not nationalisation in the normal sense of buying out the private shareholder to create a national monopoly; it was rather the creation of a co-ordinated system in which municipalities and provinces had an interest and a large measure of autonomy. The organisation set up by this Law was explained in an article by Dr. Erwin Königshofer in our February 1953 issue, but we may mention briefly that apart from certain private industrial plants and the plants serving the railways the organisation embraces four types of company—all of them joint-stock companies—municipal companies responsible for supply to certain cities, provincial companies responsible for the local supply in each of the Austrian Provinces except Burgenland, special companies formed to develop power resources of larger capacity than is needed locally, and the Österreichische Elektrizitätswirtschafts, or Verbundgesellschaft as it is termed. The Verbundgesellschaft is responsible for the new 220 kV grid, buys the power from the special companies, supplies this power where needed, and in many other respects exercises a co-ordinating and controlling role.

This organisation has functioned very successfully, and in addition to establishing the 220 kV grid many interesting and important schemes have been brought to fruition within the past ten years, and many more are in progress or projected. To mention only a few now in operation we may cite Glockner-Kaprun, Reisseck-Kreuzeck, Lünersee, Jochenstein, Prutz Imst, and Ybbs Persenbeug, most of which have already been described in this journal. Most of these constitute notable engineering achievements (incidentally, Reisseck boasts the highest gross head in the world, at 1,771·3 m., utilised in a single stage), but apart from this certain points of general interest stand out for comment.

In the first place a careful balance is being maintained between run-of-river, storage and thermal plants. At the moment about 75% of Austria's generating capacity is hydraulic and the remainder thermal, and of this 75% run-of-river stations account for 60% and seasonal storage plants 15%. Austria's coal and natural-gas reserves are strictly limited, and in future the emphasis will naturally be on water resources, which are now about 25% developed.

Secondly, the potentialities of pumped storage both for peaking purposes and for water conservation are being kept well to the fore. One of the largest pumped-storage plants in the world is that at Lünersee, in which five 56 MVA pumping/generating sets are now in service and a sixth is to be installed. Pumped storage is also an element in the Reisseck development.

Thirdly, Austria has taken a leading part in the development of international exchanges of power, and has direct or indirect connections with Belgium, Germany, France, Holland, Italy, Luxembourg and Switzerland. Indeed, the Illwerke plants in Vorarlberg exist primarily to supply peak power to southern Germany. Austria commenced to export power as early as 1926, and now one quarter of the power exported in Europe comes from Austria.

Recovery on the scale we have indicated inevitably raised grave problems of finance. From 1947 to 1953 great assistance was received under the Marshall plan, and in 1953, after a reform of the currency, an internal loan of 667 million Austrian schillings was floated successfully. Four further internal loans were negotiated during the years 1955–1958. In spite of the success of these loans, Austria could not have hoped to finance its vitally necessary electrical development unaided, and since 1954 it has received invaluable assistance from the World Bank. Between 1954 and 1957 the World Bank has advanced five loans to Austria for power development, the total capital sum involved being 56·6 million dollars. Austria has indeed received generous assistance, but the estimate we were able to form of the standard of her hydro-electric developments and of her heavy mechanical and electrical engineering products led us to feel that Austria was proving worthy of the confidence reposed in her.

New Power Station in Uganda

PLANS to construct a new self-contained hydro-electric power station on the Nile, near Atura Ferry, have been announced by the Uganda Electricity Board. This will be situated at the Karuma Falls, where there is a drop of over 40 ft., on the right bank of the River Nile, and will provide an electricity supply to parts of Acholi, Lango and Bunyoro. The works will be situated on a rock promontory which projects from the general line of the river bank. The intake will be situated immediately above the falls, and the whole scheme will be contained in an area measuring 200 yards by 100 yards. The tailrace connecting with the river will be cut in the solid rock. Precautions will be taken to protect the works against hazards incidental to the presence of elephants, rhinoceroses, buffaloes, giraffes, etc.

An interesting feature of the power station is that it will be of the outdoor type with no superstructure. It will contain initially two 1,000 kW sets and a few years later it is intended to install a third set of 2,000 kW capacity, making 4,000 kW in all. The access road to the site will start at a point about ten miles north of Atura Ferry on the main road to Gulu and will be about nine miles long.

The site is just within the Murchison National Park and the Board are working in close collaboration with the Trustees.

It should, perhaps, be explained that the Owen Falls station is too far away to provide power to the districts mentioned. Technically it would be easy to erect 132,000 V steel pylon lines but the capital cost would be high. In the years to come, as the consumption of electricity increases substantially, the 132,000 V grid will be extended and the Karuma Falls station will pump electricity into the grid. The Board will shortly be announcing details of a self-contained scheme for the Fort Portal area.

Hydro-Electric Investment in France

THE Electricité de France anticipates a 1958 expenditure of 279 milliard francs of which only about 20 milliards are earmarked for new development. In recent years the EDF has tried to keep pace with internal industrial expansion and this has involved a capital investment out of proportion to the revenue. Additional finance has been obtained from banks and by short-term stock issues with the result that in the current budget a sum of 58 milliards is set aside for repayments of loans and bank advances.

Many of the projects in the planning stage or already under construction have had to be shelved for lack of finance, although the power which these schemes would ultimately have supplied to the network would still not have satisfied the demands of French industry in the next few years. The much-discussed Rance tidal power station is one of the victims of this policy and, of all the new hydraulic power projects, only Marckolsheim and St. Estère will be completed for the time being. Half the thermal power development has been postponed while only two of the original three nuclear power stations will be continued.

Consumption of energy in France is expected to double in about 10 years and the present rate of increase is 7·2% per annum. During the next decade, the EDF may find it impossible to meet the demand without help from abroad and importing expensive power can only aggravate a precarious financial position.

France is richly endowed with hydraulic resources and she has a prosperous, expanding economy. Future progress will be in jeopardy unless new power stations are completed and the carefully prepared projects for development become fact rather than stymied drawing-board fiction.

British Trade Mission to Canada

THE present population of Canada is a little over 17 million but at the current rate of increase it may well attain 75 million during the first quarter of the next century.

Sir William Rootes recently reported to British industry on the objects, activities and conclusions of the Dollar Exports Council Delegation which visited Canada from April 24 to May 21, 1958, under his leadership. This report contains much valuable information which should provide British industry as a whole with substantial food for thought.

It is said that the "British gift for understatement is not a public relations asset" and that although Britain is an efficient, hard-working nation in the forefront of scientific and technological progress, its industrial resources are often little known and underestimated in the Canadian market. There is now a serious unbalance of trade between Canada

and the United Kingdom; although Great Britain is the largest single overseas market available to Canadians the influx of American investment has naturally affected purchases of British capital and consumer goods.

Canadian power development, although extensive, is only in its infancy but there is a growing demand for hydro-electric energy in the opening up of new territory and industrial expansion of all kinds. Speed, availability and service are the criteria for successful selling on the Canadian market and only dynamic personal approach on the spot can ensure commercial victory for British manufacturers.

The Dollar Exports Council Delegation stresses the importance of frequent visits to Canada by senior executives and the importance of permanent qualified representation in the territory. It found a readiness to invite British tenders but a need for guidance on how and where to issue such invitations.

Portuguese Electricity Supply in 1957

A REFLECTION of the world recession of the past couple of years is to be seen in the Portuguese Government's report on the country's electricity supply for 1957, because, for the first time since 1942, the total annual production actually declined. In 1957 the total production was 2,169 GWh as against 2,176 GWh in 1956—a decrease of 0·33%. In itself this decrease is a small one, but its significance lies in the fact that for the past ten years Portuguese electricity production has been steadily increasing at rates ranging between 10% and 15% per annum.

When the figures are broken down some interesting features emerge. Public-supply production actually increased by the insignificant amount of 0·15%, but private production decreased by 7·45%. This illustrates a phenomenon, which has been noticed elsewhere, that in a time of recession public services do not immediately follow the downward trend of industry but tend to remain steady.

When hydraulic and thermal production are separated the change is even more striking. Since 1949, after the national policy was established of developing the country's water-power resources in preference to large-scale thermal development, there has been a phenomenal growth in the share taken by water power in electricity production, but in 1957 water power reduced its lead. Total hydro-electric production fell to 1,841 GWh in 1957 from 2,036 GWh in 1956—a decrease of 9·58%. This was accompanied by an increase in production from thermal stations from 140 GWh to 328 GWh, or 133·9%, and if public supply only be considered the increase was from 37·5 GWh to 227 GWh—no less than 505%. This swing-back is explained by the fact that 1957 was evidently a much drier year than 1956. According to charts published in the relevant reports, run-of-river generation fell from about 1,450 GWh to 1,135 GWh, and storage plants had to supply some 706 GWh as against 580 GWh in 1956. Flows were consistently low from June onwards, and in fact only in March did the run-of-river generation exceed that of the corresponding month of 1956.

Total installed capacity rose slightly from 974 MW to 977 MW. The whole of this admittedly insignificant increase was provided by hydro-electric plant, the capacity of which rose from 759 MW to 763 MW, and in fact there was a reduction in thermal capacity of about 250 kW. Now that the Picote station on the international section of the Douro is coming into commission and is being followed in due course by Miranda and Bemposta, it is to be expected that future reports will once again record important increases in hydro-electric capacity.

The plants (both public and private) on the Zêzere have the largest aggregate installed capacity at 316 MW, but generated only 702·6 GWh as against 824 GWh in 1956. In fact, the only watercourse to show an increase in output was the Câvado, on which the total installed capacity remained unchanged at 219 MW, but generated 568·5 GWh as against 479 GWh in 1956. This increase was furnished entirely by the HICA plants, which supplied 564·5 GWh as against 474·5 GWh in the previous year; the private plants on this watercourse showed a decrease.

The output of electricity per head of population for 1957 was 222·1 kWh.

New Development in Saskatchewan

MOST of the water-power potential in Saskatchewan is in the northern part of the province where the mining industry is concentrated. The southern region is rich in other sources of energy such as coal, oil and natural gas. A vast development known as the South Saskatchewan River project is now in the planning stage and the Canadian Government has recently agreed to provide a substantial part of the capital investment involved. When completed, this project will make another 200,000 h.p. of energy available for industrial expansion in the province.

Survey of Dominican Republic Rivers

A RESEARCH group from the University of Santo Domingo and the Dominican Republic Home Office is listing the main waterfalls in the country for possible use as hydro-electric stations. Of 50 waterfalls 29 have been found suitable for hydro-electric schemes and a detailed study of these sites is being made.

New Dam and Power Station in Iran

PRELIMINARY work has just begun on an imposing double-curvature arch dam in the Elburz Mountains, Iran. The principal function of this structure, known as the Karadj dam, will be to provide Tehran some 40 miles away, with power and a much-needed regular supply of water.

The dam will be 590 ft. high, with a crest length of 1,264 ft. and base thickness of 98 ft. An interesting feature is a concrete-lined overflow spillway, on a 45 degree slope at the right abutment, which will have a discharge capacity of 51,000 cusecs. The Karadj power station will have an output of 84,000 kW and will be used mainly to satisfy peak-load demands from the Tehran area. More than 960,000 cu. yards of concrete will be laid and the excavation work will include removal of some 370,000 cu. yards of rock.

This scheme is one of several contracts awarded by the Persian Government to the Morrison-Knudsen Company Inc. The Harza Engineering Company are the consulting Engineers for this project and were responsible for the design.

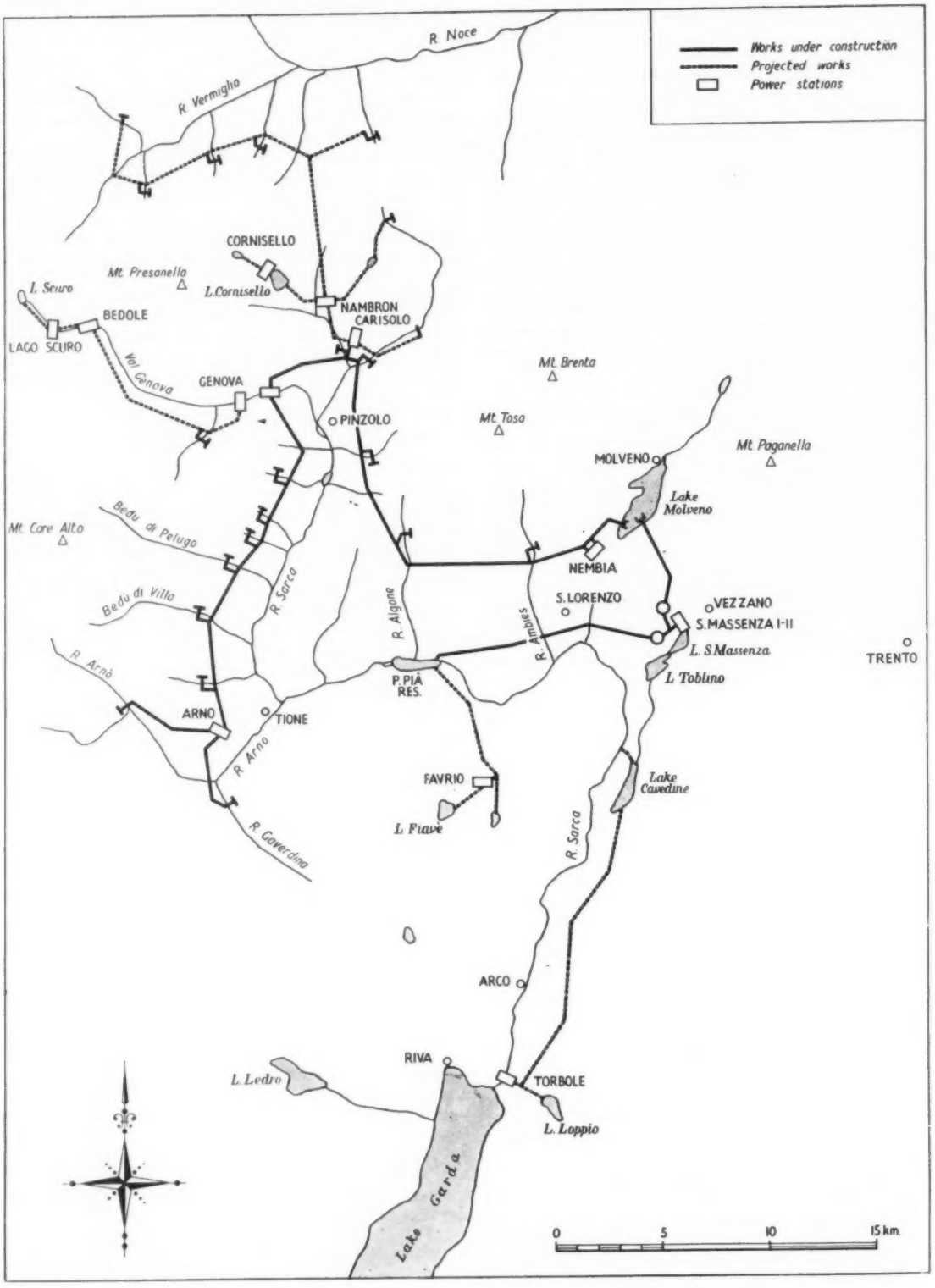


Fig. 1. Sketch map of the Sarca Molveno development

Fig. 2. Lake Molveno drained for the construction of the intakes. The pontoons carrying the drainage pumps can be seen in the left foreground

The Sarca Molveno Scheme

A co-ordinated scheme for the development of the Sarca river in the Italian province of Trentino, using Lake Molveno as seasonal storage, will have an ultimate installed capacity of 655 MVA and an average annual output of 1,400 million kWh. Santa Massenza station, already completed, has a capacity of 388 MVA and is the largest station in Italy

PART ONE

THE river Sarca rises in northern Italy in the territory between the Alps and the Dolomites, flows generally southwards between the Chiese to the west and the Adige to the east, and discharges into the northern extremity of Lake Garda. The power potentialities of the Sarca and its tributaries are considerable, and a limited company, Società Idroelettrica Sarca-Molveno (S.I.S.M.) was formed in 1939 to exploit them. This company was floated jointly by Società Edison and by Società Idroelettrica Piemonte (S.I.P.), each of whom contributed 49% of the capital, the remaining 2% being held by Istituto Ricostruzione Industriale (I.R.I.).

A map of the development scheme is reproduced in Fig. 1. The scheme will comprise ten power stations, which will have an ultimate installed capacity of 655 MW and a firm capacity of 584 MW, the estimated average annual production being 1,426·4 million kWh, of which 637·3 million kWh will be available from mid-November to mid-April.

An unusually interesting feature of the scheme is the use of Lake Molveno as the main seasonal storage. This lake was formed by a landslide which blocked the valley to a height of 125 m., and has no natural surface outlet, the hydrological balance being maintained by evaporation and underground leakage.

TABLE I.—POWER STATIONS OF SARCA-MOLVENO SCHEME

Power stations	Reservoir or intake			Average discharge	Average head	Average nominal power	Installed power	Average yearly output		
	Name	Water level	Useful storage					Winter 11/9–10/4	Summer	Total
		m.	$10^6 m^3$	m^3/sec.	m.	kW	kVA	10^6 kWh/year		
Lake Scuro	Lake Scuro	2,674	1·500	0·051	809	548	2,000	2·6	—	2·6
Bedole	Sarca di Genova	1,851	—	1·385	260	3,530	6,000	2·5	19·2	21·7
Genova	Sarca di Genova	1,555	—	2·625	650	23,100	35,000	22·0	122·4	144·4
Cornisello	Lake Vedretta	2,616	2·400	0·114	444	496	1,800	2·4	0·9	3·3
Nambron I	Lake Cornisello	2,145	15·720	1·090	768	8,207	26,000	31·5	20·4	51·9
Nambron II	Lakes Gelato, Serodoli, Nero, Ritorto	2,100	7·300	0·314	717	2,207	12,000	13·9	—	13·9
Carisolo I	Sarca di Nambrone	1,360	—	1·950	473	9,043	10,000	6·5	30·8	37·3
Carisolo II	Sarca di Campiglio	1,175	—	1·910	287	5,374	24,000	34·5	26·5	61·0
Favrio	Lake Fiavè	660	10·750	0·450	154·45	681	2,000	4·3	—	4·3
Nembia	Molveno Supply Tunnel	897	—	12·15	55·18	6,577	14,000	12·8	26·2	39·0
Santa Massenza I ...	Lake Molveno	840	207·5–234·0	15·574	553	88,738	355,000	320·0	317·0	637·0
Santa Massenza I ...	Bondai Spring	—	—	(0·956)	459·10	—	—	—	—	—
Santa Massenza II ...	Ponte Pià	462	1·0	8·35	202·70	16,591	44,000	36·8	71·2	108·0
Torbole	Lakes Toblino—Santa Massenza Cavedine—Loppio	242 221	— 18·0	29·286 0·301	166·18 154·53	47,713 456	135,000 —	147·5 —	154·5 —	302·0 —
Total						213,261	655,800	637·3	789·1	1,426·0
Reduction of energy from existing plants								50·0	140·0	190·0
Net output								587·3	649·1	1,236·4

The natural drainage area is 64·6 sq. km. and the natural storage is of the order of 182 million cu. m. at a surface elevation of 825 m.

Investigations showed that raising the storage level to 833 m. would increase the storage to 207·5 million cu. m., and by raising it further to 840 m. the total storage would be 234 million cu. m.

The feasibility of raising the storage, however, depended upon several factors. In the first place, a limit of 840 m. was set by the fact that flooding beyond this elevation would involve the hamlet of Molveno, a national roadway, and other important works. Secondly, the success of such a procedure was contingent on the ability to control the natural leakage within acceptable limits; obviously an increase in level would increase this leakage. Thirdly, the natural inflow was insufficient to maintain a higher level or to meet the requirements of a power plant, and it was essential to find some means of supplementing the supply.

To meet this last condition the scheme, therefore, is based primarily on the diversion of the waters of the upper Sarca to Lake Molveno, power being developed at convenient points in a series of stations utilising the heads between the respective water sources and the lake. This scheme is set out in detail in Fig. 1 and in Table 1, andit will suffice to mention its major features.

The Sarca is derived from three branches—Sarca di Genova, Sarca di Nambrone, and Sarca di Campiglio—and the waters from all these branches are collected in a free-flow tunnel leading to Lake Molveno. Later, each branch will be developed by a series of power stations between its source and its intake to this tunnel.

Below the confluence of these three branches a further length of free-flow tunnel retrieves the headwaters of the right-bank tributaries of the Sarca down to the Arno and conducts them to a compensating basin forming part of the Sarca de Genova intake works. In the main section of tunnel leading to Lake Molveno, two side intakes divert the headwaters of left-bank tributaries of the middle Sarca. The headwaters of the Noce, which lies north of the Sarca catchment and is a tributary of the Adige, will also later be diverted to the Sarca system via the Sarca di Nambrone.

In all, the main free-flow tunnel is nearly 50 km. long and ranges in section from 4·5 to 21 sq. m., to handle flows from 6 to 48 cu. m. per sec. Before reaching lake Molveno the tunnel bifurcates so that it can deliver to a station at Nembia which will utilise the head between the intake works and the lake. On account of the fluctuation in level that occurs due to the use of the lake as a seasonal reservoir, Nembia station will be called upon to operate on a wide range of heads—from 22 to 88 m.—and to meet this condition it is being equipped with a 12 MW Kaplan set, consisting of a Franco Tosi turbine and a T.I.B.B. alternator.

From Lake Molveno a 5 km. pressure tunnel leads to Santa Massenza station, which is the principal station in the scheme and is the largest in Italy. It has a total capacity of 388 MVA and can be used either for base load or for peak load and frequency control.

Actually, Santa Massenza is a double-head station, as 27 MW of its capacity—designated "Santa Massenza II"—utilises the waters of the middle Sarca. A reservoir of 1½ million cu. m. live storage has been formed in the river valley at Ponte Pià, and a tunnel 11·5 km. long and 3 m. in diameter, capable of passing 15 cu. m. per sec., feeds a vertical Francis set at Santa Massenza. To enable full use to be made of the storage capacity of Lake Molveno during the spring floods, pumps are to be installed at Santa Massenza to transfer surplus water from Ponte Pià reservoir to the lake.

Santa Massenza station discharges into the twin lakes Massenza and Toblino, and the final stage of

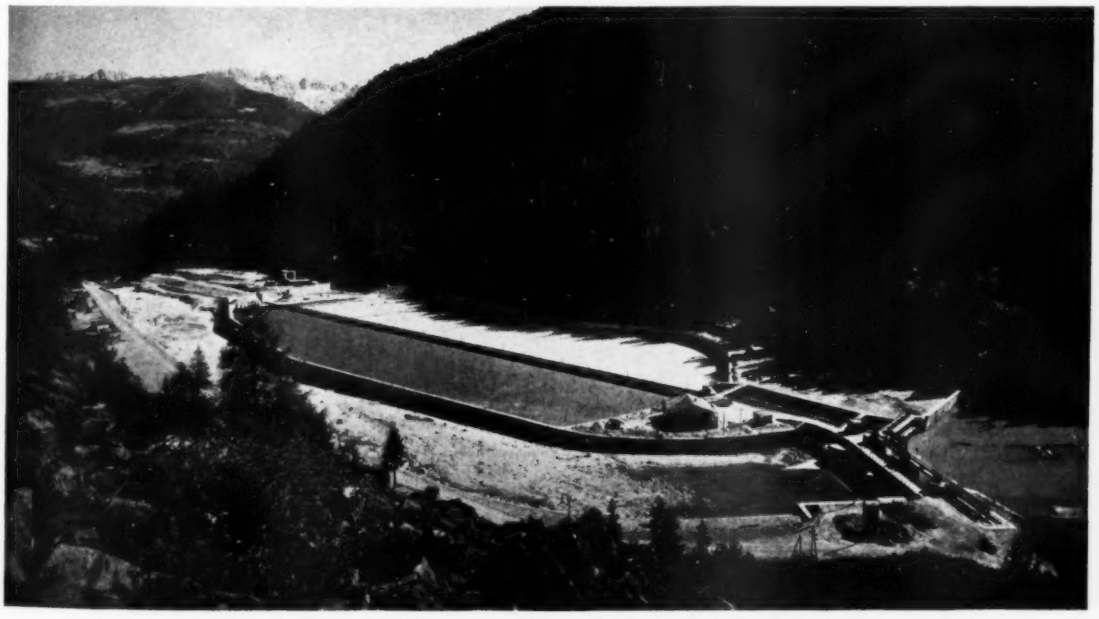


Fig. 3. Sarca di Genova intake works and compensating basin

the development will be a station at Torbole discharging into Lake Garda.

At the time of writing, the scheme is about half completed. Two 35 MVA sets in Santa Massenza station operated for the first time in March 1952 and the station is now equipped to full capacity. The main free-flow tunnel to Lake Molveno is in service and Nembia station is under construction. Ponte Pià dam is completed. The remainder of the scheme is in the earlier stages of construction or is still in the project stage. It is hoped to complete the works by about 1962, by which time the Sarca catchment will have been fully developed between an elevation of 2674 m. (that of Lake Scuro) and 64 m. at Lake Garda.

Intake Works

No less than 14 intake works have been required for the main tunnel system above Lake Molveno, and when the upper Sarca tributaries are developed a number more will be called for. These are in addition to the intakes for Santa Massenza station and the future station at Torbole.

Of the 14 supplying the main tunnel the major structures are those collecting the waters of the Sarca di Genova, Sarca di Nambrone and Sarca di Campiglio, and of these the principal one is that of Sarca di Genova.

This structure is depicted in Fig. 3. A weir, equipped with two 11·5 by 2·7 m. sluice gates, dams the river to a normal level of 897·5 m. and a maximum level of 900 m. To the right of this weir are the intake sluices, of which there are six, 3 m. wide by 1 m. high, designed for a total intake of 22 cu. m. per sec., leading to three desilting basins where fine material settles and is sluiced away. Shingle settles on the river bed in front of these sluices and is dealt with by six scour conduits, controlled by 3 by 0·6 m. sluices and discharging down river. A winter intake, controlled by a 1·1 by 1·1 m. sluice, is also provided to draw water at depth when the river is icebound.

As already mentioned, the Sarca di Genova intake also handles the waters from the right-bank tributaries of the middle Sarca, and to receive this supply the structure incorporates a compensating basin of 250,000 cu. m. capacity. A bypass channel extends across the centre of this basin to carry the water from the Sarca di Genova intake straight to the tunnel intake, which takes the form of an inverted syphon below the river bed, and is controlled by three 1 m. by 2·5 m. sluices. Near the tunnel intake this channel is joined by a smaller channel carrying up to 11 cu. m. per sec. from the right-bank tributaries. Either of these supplies can be diverted into the compensating basin, and water from this basin can be fed into the channel leading to the tunnel intake by a series of three 1 m. by 2·5 m. sluices. Above a level of 892·9 m. the bypass channel becomes drowned, and thereafter the basin can fill to a level of 896·5 m., when it discharges over a spillway into the river below the weir. The presence of the bypass channel enables the compensating basin to be cleaned out, when necessary, without interfering with the flow from the river to the tunnel intake.

About 3 km. downstream of the Sarca di Genova intake the supply is received from the Sarca di Nambrone intake, followed almost immediately by that from the Sarca di Campiglio. Both intake structures are similar in principle to that of Sarca di Genova just described, but they are free from the complication of the compensating basin necessary in that case. Each consists of a weir across the river which diverts the water through a set of sluices on the right bank to desilting basins, scour conduits being also provided to remove any accumulation of shingle from the front

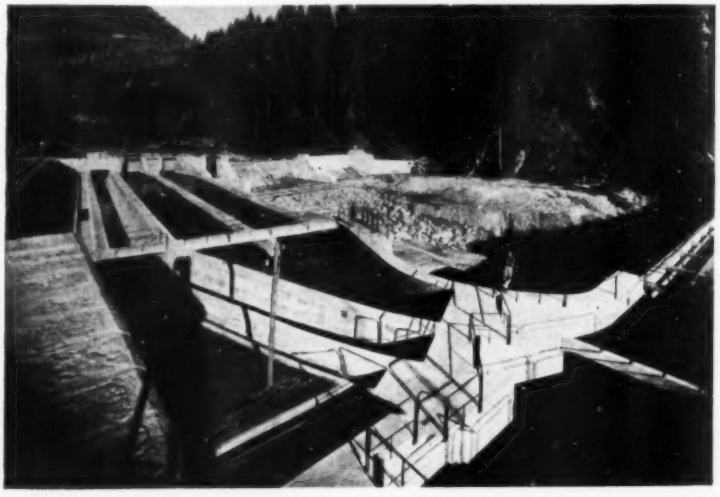


Fig. 4. Sarca di Nambrone intake works

Fig. 5. Sarca di Nambrone aqueduct bridge

of the intake sluices as at Sarca di Genova.

At the Sarca di Nambrone works the intake structure also incorporates a channel to receive the tailwater from the future Carisolo power station, and the total supply, up to 14 cu. m. per sec., is conducted to a forebay where it joins the waters from the Sarca di Genova intake works. This forebay conducts the combined waters, corresponding to a maximum flow of 37 cu. m. per sec., to a single-arch reinforced-concrete aqueduct spanning the river. Views of the intake structure and of the aqueduct appear in Figs. 4 and 5.

The Sarca di Campiglio intake (Fig. 6) is designed for a maximum flow of 8 cu. m. per sec., and leads to a connecting tunnel which joins the main tunnel between the two branches of the river. Thence the combined waters, now corresponding to a maximum flow of 42 cu. m. per sec., cross the Sarca di Campiglio by the triple-arch reinforced-concrete aqueduct, about 90 m. long, seen in Fig. 7.

In constructing the free-flow tunnel from these intake works to Lake Molveno, some difficulty was encountered when crossing the bed of the Algone, a left-bank tributary of the Sarca. The tunnel was being driven 60 m. below the present river bed, but the river was found to have formed a deep ravine between two walls of rock 20 m. apart, filled with loose detritus rich in lime and heavily waterlogged. To drive through this barrier the unstable ground was frozen by means of brine pipes inserted horizontally through the mass round the tunnel profile, and the material was removed by oil-pressure-operated splitters assisted by light explosive charges.

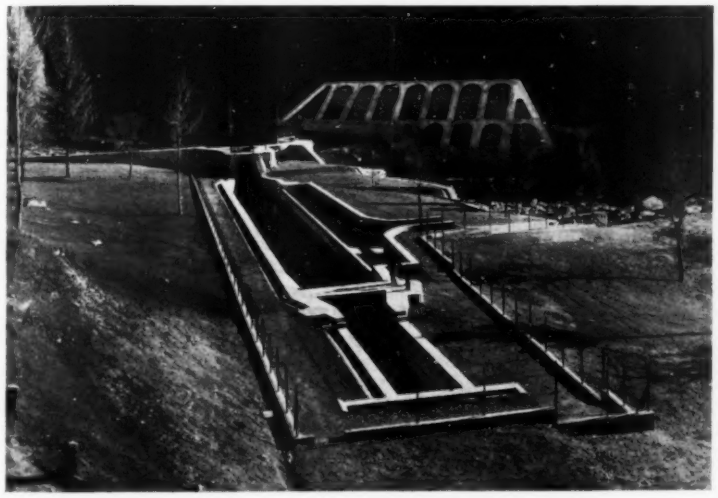


Fig. 6. Sarca di Campiglio intake works

Fig. 7. Sarca di Campiglio aqueduct bridge

Molveno Santa-Massenza Intake

The intake works on Lake Molveno to supply Santa Massenza station are of outstanding interest because of the technical problem that was set and the manner in which it was solved. We have already explained that Lake Molveno is a natural lake formed by a landslide and has no surface outlet. Its maximum natural surface level was 825 m. and it was proposed to raise this to 840 m. On the other hand, to make the utmost use of the storage capacity of the lake, it was desired to construct the intake at a sill level of 709 m., corresponding to a depth of immersion of 116 m. at natural level and 124 m. at full storage level. Underwater connections to lakes have been made successfully in certain instances by tunnelling nearly through to the wall of the basin and then blasting out the rock plug, as, for example, at Loch Fannich in Scotland, but at this depth of immersion such a procedure would have been hazardous even if the geological conditions had admitted of it. Cofferdamming or underwater working at this depth would have been impossible, and S.I.S.M. decided that the only practical solution was to drain the lake.

This proposal immediately raised the question of the disposal of the water, and it was decided to use it to good purpose by passing it through Santa Massenza station, the construction of which was to be advanced sufficiently to enable it to utilise the water for power production. To this end, the supply tunnel, surge tanks and penstocks were completed and two 35 MVA generating sets were installed at Santa Massenza before the intake works were constructed.

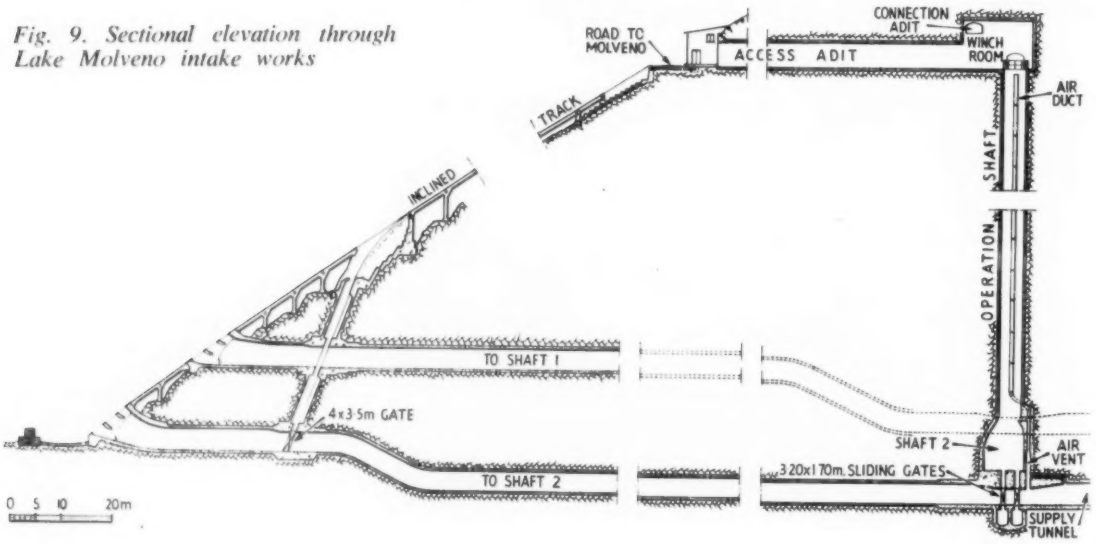
The method whereby the lake water was transferred to the supply tunnel and the intake was built is unusually interesting. Two blind shafts, subsequently to be used for the intake sluices, were sunk to the level of the supply tunnel, and from the bottom of each shaft a branch was taken, the two branches converging to form the commencement of the supply tunnel. In each shaft the downstream sluice was installed and an opening left for the upstream sluice.

Fig. 8. Lake Molveno intakes nearing completion

At the beginning of March 1952 the lake level stood at 820 m., and an adit was driven at this level from the lakeside to shaft No. 2. Meanwhile, two floating pontoons were erected on the lake, each carrying two vertical pumps, and flexible delivery pipes were run from the pontoons through this adit to discharge into No. 2 shaft, the water passing down through the opening for the upstream sluice and through the downstream sluice into the supply tunnel. These pumps were built by Costruzioni Meccaniche Riva; each was driven by a 1,250 kW motor and delivered 3·5 to 1·6 cu. m. per sec. at heads ranging from 10 to 40 m.

As the water surface was lowered it became possible to drive a second adit at 815 m., the delivery pipes being transferred thereto, and work proceeded in shaft No. 1 on an adit at 801 m. In due course, the delivery pipes were transferred to the new adit, and the water was discharged down No. 1 shaft, enabling work to be commenced in No. 2 shaft on a further adit at 788 m. This kind of procedure continued until it was possible to construct No. 1 intake structure at 728 m. and No. 2 intake immediately below it at 709 m. Some of the deeper adits, and No. 2 intake, were more conveniently driven from an

Fig. 9. Sectional elevation through Lake Molveno intake works



inclined access tunnel which had been constructed to drive the supply tunnel.

The emptying of the lake, involving the withdrawal of 200 million cu. m. of water, took nine months, and the two 35 MVA sets in Santa Massenza station were supplied by the pumps from March 30, 1952, to April 15, 1953.

To improve the watertightness of the landslide which formed the lake a double row of holes was drilled the whole length of the landslide and grouted with a mixture of cement, bentonite and sodium silicate to form a watertight curtain. Seepage was greatly reduced, and the work has been continued to allow for the raising of the water level of the lake. Any residual seepage will be collected at the foot of the landslide and pumped back into the reservoir by pumps installed in Nembia power station.

A general view of the workings after the lake had been emptied appears in Fig. 2 and clearly shows the pontoons and the flexible delivery pipes disappearing into one of the adits. The permanent intakes can be seen under construction, together with an inclined track which enables a wheeled stoplog to be lowered over the mouth of either intake by a winch located above maximum lake level. A closer view of the intakes is given in Fig 8. and a sectional drawing in Fig. 9.

The use of two intake passages, quite apart from the constructional convenience as outlined above, presents important design and operating advantages. In the first place, the sluices have to withstand a high head, and by dividing the flow it has been possible to use smaller sluices. Secondly, it is possible to isolate either intake for inspection or maintenance while the other remains in operation.

The sliding gates at the bottom of the intake shafts were constructed by Franco Tosi, and consist of plane sliding diaphragms 1·7 m. by 3·2 m. in size and operated by oil servomotors supplied by an oil pumping installation at the head of each shaft. Closure takes about 12 min.

Supply Tunnel, Surge Chamber, and Penstocks

From the intake works at Lake Molveno a pressure tunnel extends a distance of 5,230 m. to the surge chamber. This tunnel is 5 m. in diameter and is designed for a maximum flow of 70 cu. m. per sec. It operates under a maximum pressure of 16 kg. per sq. cm. and is concrete lined throughout. No permanent steel support was required, as the tunnel passes through good-quality rock in limestone and dolomite.

Driving operations were carried out from an inclined construction shaft near the Lake Molveno intake, an intermediate adit at Gaggi, and an adit at the surge chamber. Two methods of tunnel lining were adopted. For a length of 1,500 m. from the Molveno shaft circular steel shuttering was employed, and the lining was placed round the complete section by a small-capacity pneumatic placer. For the remainder of the tunnel, which was worked from the Gaggi adit, the invert was placed first and afterwards telescopic steel shuttering was erected to place the walls and roof, the concrete being delivered by a large-capacity piston pump.

At one point the route of the tunnel passed through a natural cavern, and this was bridged by a free-standing steel conduit 4·8 m. in diameter and 19 m. long.

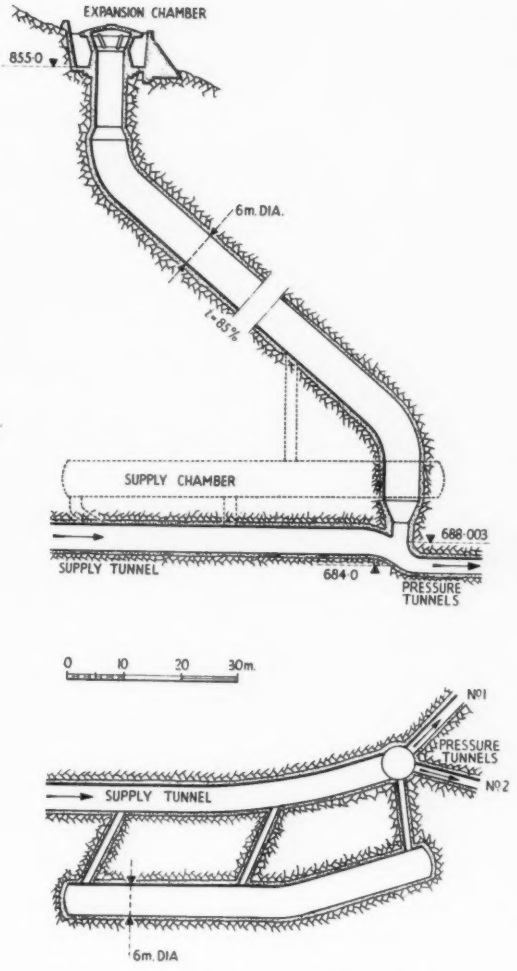


Fig. 10. Sectional elevation and plan of the surge chamber for Santa Massenza I

Details of the surge chamber are given in Fig 10, and description is unnecessary. Immediately downstream of the surge chamber, the tunnel bifurcates and continues to the valve chamber, whence two penstock shafts descend to the Santa Massenza underground power house. The shafts were driven from the bottom and from an intermediate adit.

The valves at the penstock heads were supplied by Calzoni and are of the cylindrical-spherical rotative type, with 2,100 mm. bore. Sealing is normally by means of a downstream sealing ring, but should this fail to function efficiently, the valve body can be displaced upstream by a servomotor against an emergency sealing ring. Each valve is served by an oil pumping set with pneumatic accumulator of sufficient capacity to close the valve even in the event of electrical failure. Each valve with its accessories weighs over 70 tons, and takes about 40 sec. to operate.

The penstocks consist of steel tubes ranging in diameter from 2·6 to 2·4 m. and in wall thickness from 16 to 48 mm., and solidly encased in concrete. No. 1

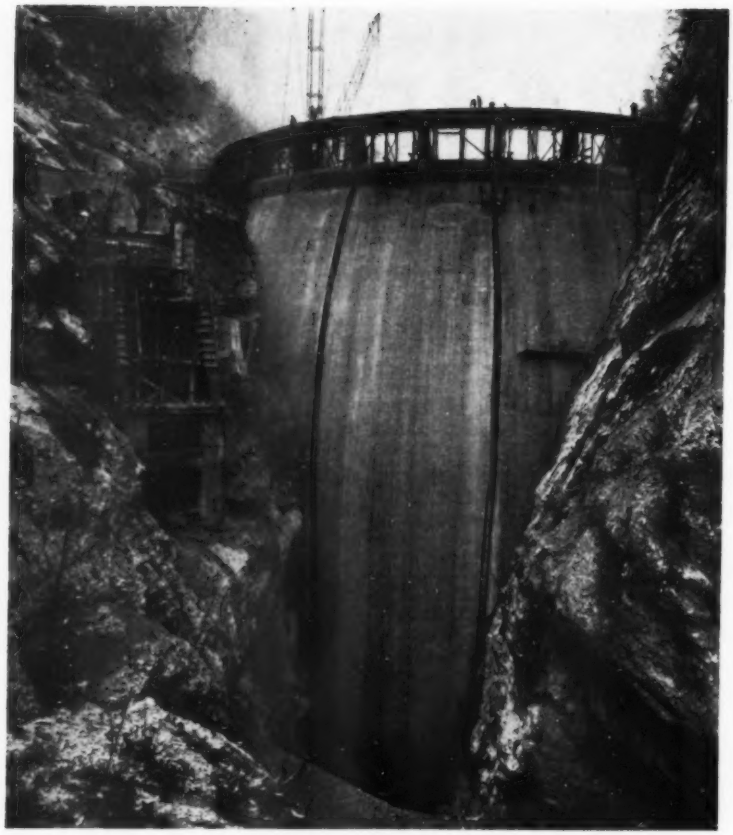


Fig. 11. Ponte Pià dam approaching completion

penstock is 733 m. long and No. 2 is 834 m. long. These penstocks are claimed as an outstanding technical achievement in view of their dimensions and weight and of the fact that they are the first installation in Italy using special extra-high-tensile steel.

Ponte Pià Reservoir

Ponte Pià reservoir, which, as already mentioned, is located in the middle Sarca and supplies Santa Massenza II, has a live storage of 1½ million cu. m. and a total storage of 3½ million cu. m. It has been impounded by a double-curvature concrete arch dam 54 m. high above lowest foundation and 46 m. high above the river bed. The dam has a crest length of 58 m. excluding the abutments. The thickness ranges from 4·85 m. at the base to 2·0 m. at the crest, the thickness at any horizontal section being constant. The horizontal radius of curvature ranges from 23·2 to 14·0 m. and the subtended angle from 140° to 108°.

The crest arch is expanded, steel reinforced, and shaped to provide an overflow sill, which comprises six openings extending nearly the full width of the crest. This sill is for emergency operation only, and for normal use there is a spillway discharging into a free-flow tunnel driven in the right bank, this spillway being controlled by a Galileo sluice gate, with superimposed automatic flap, 9 m. wide by 8 m. high overall. There are also an intermediate and a bottom discharge. The inlet mouth of the intermediate discharge is located below the intake structure, and is controlled by two 2·9 m. by 1·75 m. sliding gates in series. The bottom discharge consists of a separate tunnel controlled by four 2·9 m. by 1·75 m. sliding gates, comprising two pairs in series. All the gates for the intermediate and bottom discharges are of Calzoni manufacture. The intake is equipped with an ATB circular sluice gate 3 m. in diameter. All the gates are oil-pressure operated from a central oil pumping and storage plant.

The supply tunnel to S. Massenza II is about 11·5 km. long and 3 m. in diameter, designed for a maximum head of 5 kg. per sq. cm. It is followed by a steel penstock entirely encased in concrete in the rock, having a diameter of 2·4 m. and a thickness ranging from 8 to 20 mm. The penstock inlet gates, which are of ATB manufacture, are of the wagon type, 1·9 m. by 2·4 m., and are operated by oil servomotors.

The intake arrangements will handle a flow of 15·5 cu. m. per sec. at the full turbine discharge, but provision is being made for a future increase of the flow to 18 cu. m. per sec. to allow for the night pumping of water from Ponte Pià reservoir to Lake Molveno during the spring floods. A view of Ponte Pià dam nearing completion appears in Fig. 11.

(To be continued)

Weldmesh Erosion Crates. Erosion crates or gabions are useful for building dykes, groynes, weirs and retaining walls to prevent erosion, and to assist in the control of storm water. The British Reinforced Concrete Engineering Co. Ltd. manufacture crates made up from welded steel wire mesh of suitable aperture and gauge into cages of convenient size to suit the particular conditions; they are supplied in collapsed form, can readily be erected at site and filled with stone.

An advantage in this method of construction is the permeability of the crates which, when filled, permit the entry and drainage of water in them. An important feature has been the use of foundation mattresses (a form of crate) which are laid on the river bed and then filled. The mattresses settle down and conform to the uneven surface thus giving a rigid and level foundation for the walls. Weldmesh is an electrically cross-welded steel wire mesh made by B.R.C. Engineering Co. Ltd. It is available in a very wide range of gauges and mesh sizes and comes in sheets or rolls. It is specially suited for erosion crate construction because it is flexible, efficient and economical.

Priest Rapids Development. In referring to the turbine and generator shafts for this power station in our August issue the address of Ruhrstahl A.G. should have been given as Henrichshutte, Hattingen/Ruhr, and not Essen.

Fig. 1. The explosive correctly distributed and detonated must break up the solid rock into small pieces but leave nearby houses intact

Ground Vibrations in Blasting

This article deals with the measurement and character of ground vibrations caused by blasting and makes recommendations by which the intensity of the vibrations can be reduced and damage to structures or property avoided

By U. LANGEFORS*, H. WESTERBERG* and B. KIHLSTRÖM*

PART ONE

WITH the development of rock-blasting technique the question of ground vibrations has become increasingly important. In Sweden, where many densely populated districts rest on rock, heavy charges are being increasingly used in city areas. This has become possible by the adoption of multiple-row blasting with short-delay ignition[9], by new methods for calculating the charge required to ensure loosening of the rock without unnecessary throw,[13] and by increased experience concerning ground vibrations and how these can be reduced in large rounds.[5] In many cases where blasting work has to be carried out in the neighbourhood of buildings or existing structures the ground vibration is the factor which decides how the blasting is to be carried out. Experience from records and investigations which have been made by Nitroglycerin A.B., the Street Department of the City of Stockholm and Skånska Cement A.B. during a

* Nitroglycerin A.B. and Swedish Detonation Research Foundation, Stockholm.

long sequence of years has now been collected and compiled so that it can be directly applied in the planning of blasting operations.

A great difficulty when it comes to determining the boundary values for varying degrees of damage is due to the fact that there have been relatively few cases where damage could be proved. As a rule every care has been taken to avoid damage. It is questionable, however, if this is not an erroneous attitude. It sometimes entails great expense to impose heavy restrictions on the carrying out of blasting operations, whereas the damage which is avoided by doing so, e.g. the falling of plaster or occasional cracks in repeated blasts, could have been repaired or compensated at a moderate cost. When there is no risk of personal injuries it may therefore be right to risk minor damage and to carry out the work in a more satisfactory manner from a technical and economic point of view.

The adoption of this principle has resulted in fine cracks and falls of plaster taking place in the course of the past few years in Stockholm, where the radical replanning of Lower Norrmalm is being undertaken and where the buildings surrounding a blasting site are due for demolition in any case. All data concerning charge, firing sequence and distance are carefully recorded and the ground vibrations accurately measured during this work.

Apparatus for Recording Ground Vibrations

In order to ascertain if the ground vibration could be reduced by interference in successive initiation of the charges, an investigation was begun by Nitroglycerin AB in 1946 at different blasting places. As the vibrographs then on the market could not reproduce the actual range of frequencies in a satisfactory manner, an apparatus was constructed in collaboration with Svenska AB Philips comprising a capacitive detector with a natural frequency of 2 c.p.s. and a cathode-ray oscillograph.[5] In order to obtain sufficient dissolution at the highest frequencies and avoid an unnecessarily large and expensive consumption of film, a special arrangement was introduced for the photographic recording. In this the oscillograph is utilised in the ordinary manner with repeated deflections from left to right, and the film of the camera is fed downwards at a constant speed. The recording takes place with parallel lines almost perpendicular to the longitudinal direction of the film as a base line. The sensitivity and the recording speed can easily be varied within wide limits, and this has proved to be a great advantage.

The recordings obtained with this apparatus indicated that an interval of the order of centiseconds might reduce the ground vibration by interference and also by the distribution of the charge in respect of time. The results of the experiments served as a basis for methods in short-delay blasting which have been tried out in Sweden.[5, 9, 11]

An adaptation of the so-called Cambridge vibrograph that is not only independent of electric current but also easily transportable, has been used by us for some years. The instrument has a swinging mass with relative damping and the vibrogram is drawn in a moderate enlargement on a celluloid strip. The proper frequency is 4 c.p.s. and the instrument gives vibrograms with a legible dissolution up to 500 c.p.s.

In many cases the ground vibrations which we have determined have frequencies which are many times larger than those which have been measured elsewhere. The question of the origin of ground vibrations is so complex that we will not go into it here. We would only point out that we have asked ourselves whether these high frequencies could be due to natural vibrations in the instrument. If that were so it should be possible to discover one or more permanently recurring frequencies, but this has not been the case. Furthermore, it should be possible to produce the eventual natural frequencies by jolts, but when a change is made in the testing conditions recurring frequencies are never obtained. In addition, when recording simultaneously with two entirely different vibrating systems, as with the old and present measuring equipments, we have obtained the same graph in the actual frequency area.

The Cambridge vibrograph is fitted with a vibrating system with relative friction damping (Fig. 2). If the base executes a constant sine oscillation with the amplitude A and the frequency f the spring-suspended mass will move slightly in relation to the stand, and there will be obtained for the elongation of the spring

$$-\xi = -p\sin\omega t + x \qquad \ldots (1)$$

in which $\omega = 2\pi f$ and x is the distance of the mass from the position of equilibrium. The magnitude ξ is recorded graphically on a celluloid strip which is continuously fed forward by a needle with a rounded point, the movement of which in relation to the strip is in proportion to ξ. The damping force is assumed to be

$$F = f_1\frac{d\xi}{dt} + f_2 \qquad \ldots (2)$$

in which f_1 represents 'viscous' damping, f_2 'frictional' damping. For the damping coefficients, the relationships f_1 = constant and f_2 = constant apply; f_2 changes its sign with the direction of movement so that the movement is counteracted. The relative movement is determined by equating the sum of the inertia force of the mass, the damping force, and the tension in the spring, to zero.

$$-m\frac{d^2x}{dt^2} + f_1\frac{d\xi}{dt} + c\xi = -f_2 \qquad \ldots (3)$$

or

$$\frac{d^2\xi}{dt^2} + 2b\frac{d\xi}{dt} + \omega_o^2 = \omega^2 p\sin\omega t + h\omega_o^2 \qquad \ldots (4)$$

where m is the spring-suspended mass, c the spring constant, $f_1/m = 2b$, $c/m = \omega_o^2$ and $-f_2/m = h\omega_o^2$.

To determine the magnitudes b, ω_o and h, the natural oscillations of the system can be studied according to eq. (4) with $p = 0$. As long as the needle moves in one direction, e.g. upwards, h is constant and the movement follows the equation

$$\frac{d^2(\xi+h)}{dt^2} + 2b\frac{d(\xi+h)}{dt} + \omega_o^2(\xi+h) = 0 \qquad \ldots (5)$$

which implies that $(\xi + h)$ performs an ordinary damped sine oscillation

$$\xi + h = Be^{-bt}\sin(\sqrt{\omega_o^2 - b^2}\,t + \delta) \qquad \ldots (6)$$

which assumes the value 0 for $\xi = -h$. The maximum deflection upwards then becomes $(B - h)$, which is obtained if the damping factor b can be ignored. When the movement alters its direction (6) again applies, but with a new zero for the sine function

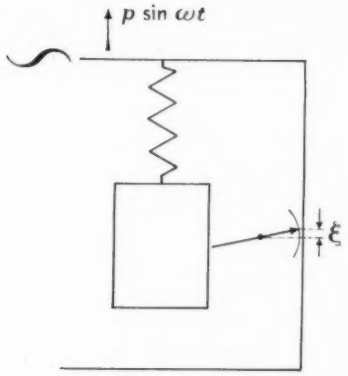


Fig. 2. Vibrograph with frictional damping

as h is now negative. To enable this to be done the amplitude at the moment of turning must be reduced to $B - 2h$ and the maximum deflection downwards will be $B - 3h$. The damping factor f_2 in (3) consequently gives a constant difference between two consecutive readings irrespective of the size of the deflection. In contrast to this the second factor f_1 or b will give a reduced amplitude which is in direct proportion to the amplitude if this is calculated from the zero of the sine function at the actual moment.

Examinations of the natural vibration of the apparatus (Fig. 3*a*) show that the amplitude decreases by a constant amount, 0·4 mm. for every deflection. It is furthermore evident that the zero of the sine oscillation for the first downward movement lies almost on the same level as the last deflection. The upward movement has its zero at an equally great distance below the inserted middle line. Fig. 3*a* is recorded with the usual gearing of 5:1 for drawing on the celluloid strip. As it is the friction of the needle on the strip which gives the chief damping this can be diminished by reducing the gearing. In drawing with the enlargement of 1:1 a greater number of full vibrations will therefore be obtained before the natural vibration has been damped out (Fig. 3*b*). It

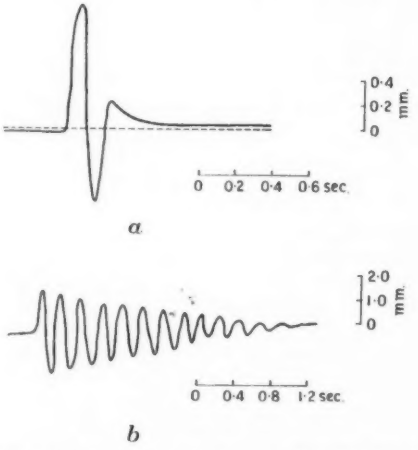


Fig. 3. Natural vibrations in the Cambridge vibrograph. a. Enlarged five times; b. without enlargement when recorded

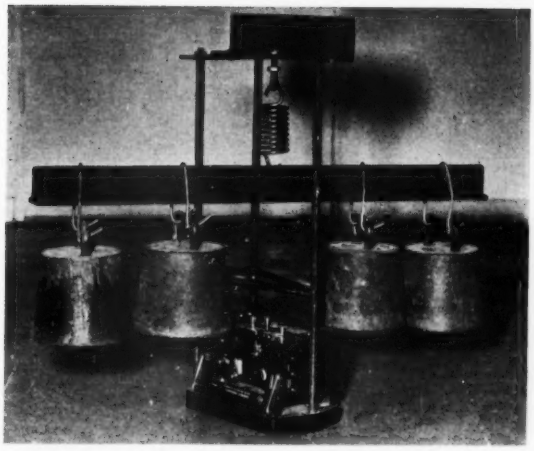


Fig. 4. The Cambridge vibrograph arranged for measuring; the instrument loaded to allow registration of vibrations with accelerations up to 10 g.

is quite obvious here that the amplitude practically decreases linearly, which implies that $b \sim 0$, and that the damping is entirely due to a constant frictional force f_2 in (2), which counteracts the movement. The decrease here is 0·08 mm. for every deflection.

The weight of the vibrograph is about 6 kg., 2 kg. of which is in the vibrating system. This means that in downward accelerations greater than 1·5g, the apparatus loses contact with the base. When measuring near blasting sites in solid rock there is often reason to calculate with higher acceleration values. When this is so it is necessary to arrange for the apparatus to maintain contact with the base throughout the entire recording. This can be done by clamping or pasting it to the base, or placing a counter force in a suitable manner on the apparatus. Mr. S. Wiklund, of PUB constructed a load arrangement, Fig. 4, which has proved effective in field work. It consists of a stand which rests on the vibrograph and sup-

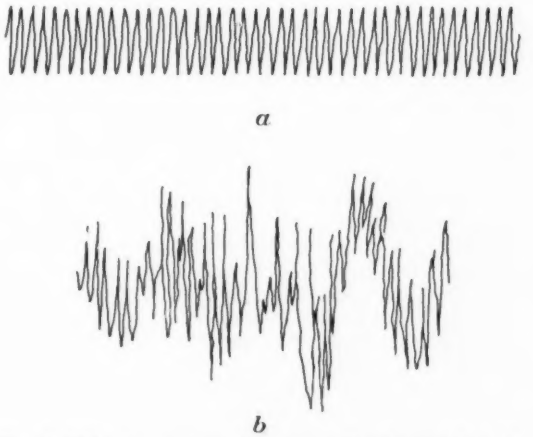


Fig. 5. Vibrogram recorded on a vibrating table with the frequency 50 c.p.s., amplitude 360 μ, i.e. at 3·6 g. a. The instrument loaded with 40 kg.; b. the instrument unloaded

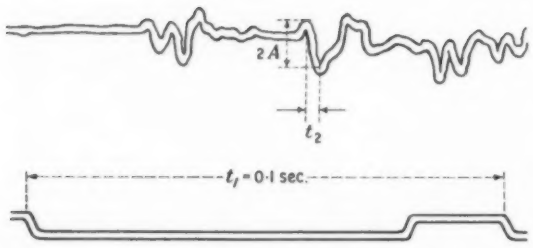


Fig. 7. Example of vibrogram reading

ports, by means of a spring, a weight which is 10 times that of the unsprung body of the instrument, i.e. 40 kg. The vibrograph should therefore give correct recording of vibrations with accelerations of up to 10*g*. Fig. 5 reproduces vibrograms which have been recorded on a shaking table with a frequency of 50 c.p.s. and the maximum acceleration 3·6*g* with a loaded (5*a*) and unloaded instrument (5*b*) respectively. It is obvious that the vibrogram in the last-mentioned case is very misleading. Fig. 6 gives an example of the difference between loaded and unloaded instruments in a single course.

The advantage of the loading arrangement is that there is no need to clamp the instrument to the base, which, as a rule, is difficult. The same effect as with the loading arrangement can be obtained if the supporting plate is simply pressed down by hand during the measuring. Satisfactory contact can then be maintained up to 4*g* and probably even in slightly higher accelerations.

In most cases the instrument is placed on the rock close to, or inside, the building, so that the best possible contact is obtained with the rock. It is consequently the vibrations of the rock that are first determined — the different types of vibrations which are obtained in walls and floors are an effect of these ground vibrations and of the special construction of the building. The question of comparing the incoming vibrations with the resulting ones may have to be considered in a second stage of an investigation.

When it is a question of determining the connection between amplitude, frequency, charge and distance, this of course has to be considered in relation to the conditions in the ground. When it is a question of indicating to the blaster the limits to the size of the vibrations it is also best, from the practical point of view, to adjust the demands so that the ground vibrations do not exceed certain stipulated values.

Reading the Vibrograms

The maximum amplitude, frequency, vibration velocity and acceleration are determined by vibrograms. Fig. 7 gives an example of how this determination is made. The vertical distance between two consecutive deflections pointing in opposite directions is determined and stands for the double amplitude, 2 *A*. If the vibrogram has been magnified the figure obtained is first divided by the magnifying factor, e.g. 5 in drawing, 10 in photographing, 50 in all. The distance horizontally is indicated by t_2. During the period of observation it is considered that the vibration can be represented with satisfactory accuracy by a sine oscillation with amplitude *A* and the period of vibration 2 t_2 for a full period. The frequency f is then obtained by comparison with the timing t_1=0·1 sec.

$$f=\frac{10\,t_1}{2\,t_2}=5\frac{t_1}{t_2}\text{c.p.s.}$$

(Continued on page 351)

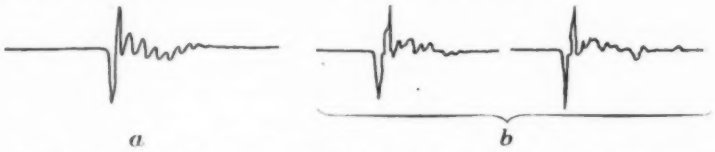


Fig. 6. Record of a hammer blow. a The instrument loaded with 40 kg.; b the instrument unloaded

Vibration Report No. 1

Blasting site: Secondary Power Station, surge chamber.

Site of vibrograph: Close to relay cabinet T 7 on the floor of the relay chamber.

Type of instrument: Cambridge Vibrograph.

Measuring done by: B. Kihlström.

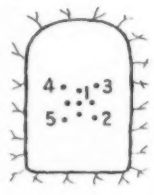
Nature and state of building and foundation: Concrete on rock.

Explosive: LFB.

Diameter of drill: 31, 30, 29 mm.

Date	Delay No.	Holes			Measuring			Results			
		Charge kg.	Number	Burden m.	d m.	h m.	r m.	A μ	f p/s	a g	v/c μ/m.
12/7 1954	1	1·2	1				38·0	10	150	0·9	1·9
,,	2	1·0	1				38·0	6	150	0·5	1·1
,,	3	1·1	1				38·0	6	150	0·5	1·1
,,	4	0·92	1				38·0	*	*		
,,	5	1·0	1				38·0	*	*		

* Values too small to be read off.



Drilling plan.

Electric detonator: ½ s.

Depth of hole: 2 m.

Fig. 8. Example of vibration report

Fig. 1 A panoramic view looking upstream

Progress at Serre Ponçon

A brief account of the stage reached in April, 1958

By PETER O. WOLF

PART TWO* of the series of articles on "The Durance Project" contained a description of the design of the great earth dam being built to harness the waters of the Upper Durance and to regulate the river flow for the benefit of both power production and irrigation in the areas downstream. Part Four concluded with a brief progress report on the stage reached in April 1957, and a comparison between Fig. 14† (taken in April 1957) and Fig. 3 (April 1958) on page 341 will give an indication of the great progress made during the past year in placing the core and the supporting fill of the dam.

Figs. 1 and 2 show a panoramic view of the fill, from a point on the Serre Ponçon rock to the west of the dam, looking upstream. Fig. 3, again taken from the observation platform on the right bank, is a view southwards along the centre-line of the dam. The rocky spur of the Serre de Monge which narrows the valley and so forms a natural dam site is clearly seen in Figs. 2 and 3.

Figs. 1 and 2 show the downstream slope and the western access road which enables the 25 cu. yd. Euclid bottom-emptying semi-trailers to reach the present top with their loads of alluvial fill obtained

* WATER POWER, Vol. 9, No. 5, May 1957, p. 177.
† WATER POWER, Vol. 9, No. 7, July 1957, p. 263.

Fig. 2. Another view showing some of the construction equipment and the rocky spur of the Serre de Monge which narrows the valley and forms a natural dam site

from the site of the balancing reservoir. The present top of the upstream slope is visible in the middle distance on Fig. 1.

Figs. 1 and 3 show that the downstream fill and the argillaceous core are approximately level, but that the transitional filter material has been placed to a greater depth. The recently placed and watered core material shows up as the dark mass in the centre foreground of Fig. 3. The various items of hauling, spreading, watering and compacting plant may be seen in Figs. 1–3.

Last year it was found that progress on the core lagged behind the placing of the supporting fill so much that the use of sheepsfoot rollers was discontinued and the thickness of deposition increased from 20 to 30 cm. Compaction of these thicker layers by ten passages of pneumatic-tyred rollers has been found to be adequate even for the argillaceous core material. There is a site soil-mechanics laboratory which maintains a close check on the material as placed, at the rate of 1 sample for every 600 cu. m. of fill.

The quantity of material placed in the dam by April 1958 represents approximately one-third of the total, and the average depth was 35 m. The best weekly average output has reached 25,000 cu. m. per day, with the Euclids working, in half-hour cycles, for two shifts per day (3 a.m. to 1 p.m. and 1 p.m. to 11 p.m.).

The total quantity of rock excavation, for the tunnels, valve house and power station, had in April reached 400,000 cu. m. Concreting of the valve house, the power station and the surge (expansion) chambers on the tailrace tunnels is now in progress.

Fig. 4 is a view, from the national highway to be submerged in the great reservoir, in a south-westerly direction, and shows the gated twin intakes to the bottom tunnels at present used for the diversion of the Durance (seen in its bouldery bed on the left of Fig. 4) and later to feed both the power and the bottom draw-off tunnels.

The wingwalls guiding the water into the bottom intakes are pierced by holes (just visible in Fig. 4) through which the diverted water of the River Blanche will ultimately be made to flow. The intake on the Blanche, approximately 4½ km. downstream of Serre Ponçon, is at elevation 800 m., and the annual flow of 100 million cu. m. entering the Serre Ponçon reservoir at a high velocity will jet alluvial deposits away from the intakes and help to keep silt in suspension, so ensuring an unobstructed flow to the turbines even after a considerable amount of bed-load has been brought into the reservoir by the Upper Durance and the Ubaye.

Fig. 3 The core material shows up as a dark mass in the foreground

The openings immediately above the gated bottom inlets are to be sealed by concrete slabs in which explosive charges are to be incorporated so that, if the gate-opening mechanism fails at a time of heavy flood inflow into the reservoir, a bypass can be opened by blasting the cover slabs.

The operating gate for the right-hand bottom intake is now being assembled (Fig. 4).

The inlet to the upper (gated) main flood spillway is seen under construction in the middle distance towards the right of Fig. 4. The tunnel for this inlet is at present under construction.

Mention may be made here of an incident which has left a deep impression on the minds of the engineers in charge of this scheme. The designers of Serre Ponçon have been conscious of the vital need for a total spillway capacity sufficient to ensure that the

(Continued on page 349)

Fig. 4 The gated twin intakes to the bottom tunnels

Fig. 1. Hungry Horse dam and 8 cu. yard Standard American cableway

Fig. 2. Lookout Point dam and 8 cu. yard Travelift cableway

Cableways for Dam Construction

By IAN B. MACKINTOSH,* M.A., A.M.I.C.E., A.M.ASCE.

THE cableway is a major item of construction plant for concrete dam building. Where cableways are not used the alternative is trestle-mounted whirley cranes or crawler cranes. The principal factor in choosing the handling method will be economy. Generally speaking, a deep narrow gorge will favour a cableway. Cableways have high first cost and require considerable skill for erection and operation.

All cableways consist of a track cable, also known as the main gut, and its supports; a carriage which runs on the track cable; and a power system for operation. The last has three functions:—

(1) To move the carriage along the track cable.
(2) To raise and lower the load.
(3) To move the towers supporting the track cable.

The power may be applied from a motor in the carriage, but more usually it is located in one of the towers. Two hoists are used, one to move the carriage and the other to lift the load.

There are three main types of cableway in which the operating ropes have different arrangements. These three types are known as the English, the Standard American and the Travelift.

English Type

The arrangements in the English type of cableway are shown in Fig. 3*a*. The carriage frame has two sheaves, one at either end, and the load hook hangs down between these. The cable for moving the carriage is fixed at each end of the carriage and runs through sheaves near the two track cable anchorages. This cable usually takes two and a half wraps around a grooved driven drum, and because it forms a closed

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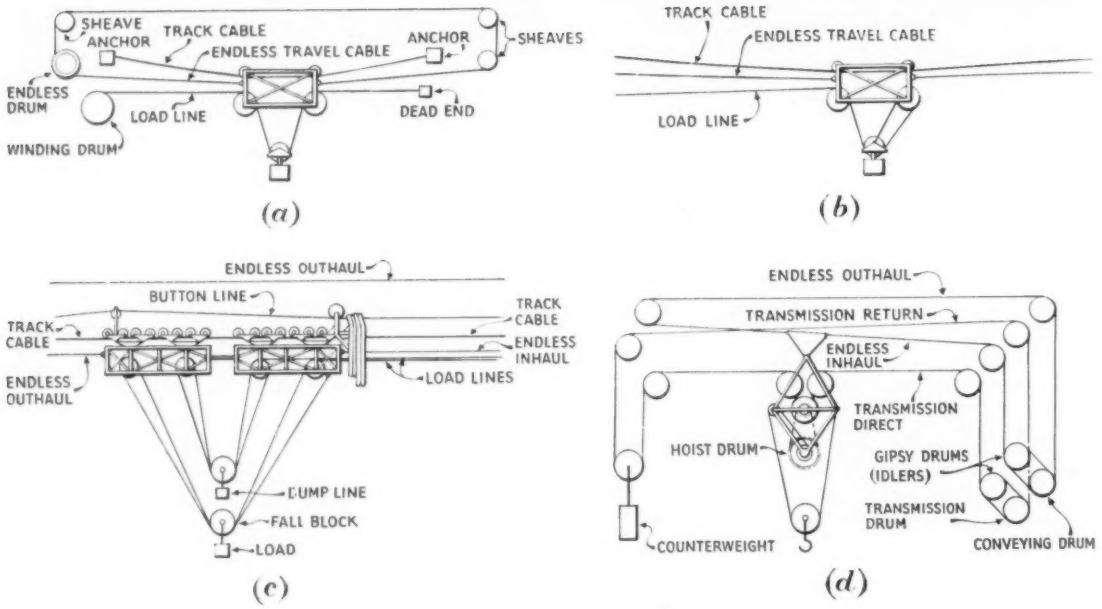

Fig. 3

(a) Single English
(b) Modified English
(c) American Standard
(d) Travelift

circuit, is called the endless cable.

The load cable is dead-ended at one of the main anchorages and is wound on to the hoist drum at the other anchorage. At the carriage it passes over a sheave at one end down and under a sheave attached to the hook known as the fall block and back up to the other carriage sheave. This system makes travelling easy because the load line pull on the carriage is balanced. However, due to continuous reversal of flexure at nearly every sheave there is very heavy wear on the load line.

In the modified English type shown in Fig. 3*b* the load line is dead-ended either on the fall block or on the carriage and there is only one flex. Furthermore, during travelling without lifting, the load line does not run through the carriage sheaves and fall block sheave as it does in the simple English type. However, in the modified English type the advantage of easy travelling is sacrificed.

In these types allowance has to be made for a large sag in the cables to prevent interference with work on the ground.

The English-type cableway is only suitable for relatively light loads and slow speeds. It has a moderate first cost.

American Standard Carriage

The essential difference between this and the modified English type is the use of slack carriers in the American type. Slack carriers are a costly feature as they involve the use of a button line to space out the carriers. The button line is a fixed line stretched between the anchorages and runs above the track cables on idlers in the carriage. Spaced out on the button line there are a series of cone-shaped buttons, each button a fraction of a size larger than the previous one. Each size corresponds to a slack carrier and the carriers are snatched off the horn on the carriage by the appropriate button. This makes the operation of this type of cableway rather jerky. Button lines will break at the buttons or connecting pins unless closely watched, and this is a source of danger to personnel.

The operating arrangement of the Standard American type of cableway using a tandem carriage is shown in Fig. 3*c*.

Travelift

This is a recent design and its purpose is to eliminate button lines and slack carriers. The impact of carriers being pulled off the horn or racked on to the horn creates the danger of a broken line and this makes carriage speeds over 1,350 ft. per minute impractical. The Travelift can travel and hoist simultaneously and at greater speed. The ratio is 7 : 1, in other words haul 70 ft. and raise and lower 10 ft. On a cableway job there is usually a narrow canyon and this ratio may be too high to be of much value. Carriage speeds range between 1,800 ft. per min. in low gear and 2,150 ft. per min. in overdrive. Experience has shown much trouble with the overdrive and it is often removed.

The arrangement of ropes is as shown in Fig. 3*d*. On the carriage is mounted a cable-powered engine operating a chain-driven load hoist. The transmission line is endless and is reeved through three transmission sheaves as shown. The hoisting line is reeved in only two parts, and so a commodious hoist drum does not have to be provided. These two parts are kept as wide apart as possible to prevent twisting of the load.

If both operating cables move at the same speed the carriage will move along without raising the load. If only the transmission cable moves, the hoisting drum

will revolve, raising or lowering the load. If only the conveying line moves, the load rises as the carriage approaches the head tower and falls as it moves away. It is possible to make the transmission line move at twice the speed of the conveying line, thereby reversing the previous effect.

Life of American Standard Type Cables

The most common size of cableway used on large dams is the 25-ton American Standard type rig which handles a skip containing 8 cu. yards of concrete. This size of cableway will be considered to show what life may be expected from the various cables involved. The lives are expressed in cu. yards output of the cableway.

Track Cable or Main Gut. A track cable is a lock-coil rope made up of a central core of round wires surrounded by two layers of wedge-shaped wires. The outer layer is formed in strands which key into one another to form a smooth running surface, thus reducing wear. A 3¼ in. diameter cable weighing 25·6 lb. per ft. has a breaking strength of 580 tons. A maximum design load of half this, say 290 tons, would be satisfactory.

The life of a track cable usually depends upon the distance between the main pin connection and the carriage location at the point of pick up for skips of concrete. A minimum distance of 300 ft. for a 2¾ in. diameter gut and 400 ft. for 3 in. to 3½ in. diameter is recommended. In nearly all cases of track cable failure, the bending of the armour wires or outer lock-coil strands, caused by load concentration at the bogey wheels of the carriage, gives rise to flexing, which in turn results in crystallisation of the wires. The bending angle, known as the T/G factor where T is the horizontal tension in this cable and G is the wheel load, is a cable manufacturer's nightmare. Very rarely has a cable failed elsewhere in advance of this location. With a fixed concrete car track it is not practicable to vary the pick-up point.

At Hungry Horse dam two main guts were used that had already seen service at Shasta dam. One of four similar 8 cu. yard cableways at Hungry Horse dam is shown in Fig. 1. At Waneta dam the gut had been used for placing all the concrete at Conemough dam. This is not unusual.

At Lookout Point dam, illustrated in Fig. 2, it was not possible to locate the pick-up point far enough from the head tower. It was about 300 ft. for a 3¼ in. cable. The armour strands started breaking after only 2 million cu. yards were placed. The armour strands do not affect the strength of the cable appreciably; the problem is to ride the carriage over the broken wires. It is very rare for a cable to break, although they have been known to pull from the jewels.

Fig. 4. Travelling head tower, Donnells dam and 8 cu. yard cableway, with concrete-type delivery car

The life of a cable is unpredictable; up to 2 million cu. yards have been placed with one cable, and as little as 300,000 cu. yards.

If a cable is taken down and rehung it must be hung in the same position as before; otherwise, flexing stresses in the armour will be reversed, causing almost immediate failure of the outer lock-coil strands.

The cableway which placed 800,000 cu. yards at Lookout Point dam was used again at Table Rock dam. The main gut from Lookout had nine broken strands and by reversing the pick-up end it could have been used. However, on a job the size of Table Rock, 1,300,000 cu. yards, it was essential to have a spare when starting with a worn cable and so a new cable was purchased. It takes three to four months to get delivery of a 3¼ in. diameter track cable. U.S. Steel spun the cable in two 8-hour shifts, but it took weeks to set the dies and jigs. The mill also had to stop other production to make the special track strands. They only spin one gut or so a year and keep no wire for this purpose in stock.

Endless Conveying Line. One end hauls in, the other hauls out.

and both ends are becketed off on the carriage. The driving drum is up to 6 ft. in diameter and the cable takes two wraps around the drum. If the tower sheaves are at least 45 diameters larger than the cable, this conveying line will easily place 2 million cu. yards plus yarding.

Hoist Cable. This cable has long life due to the large diameter of the drum, and of the sheaves in the carriage and at the head tower. It should last for from 500,000 to 1 million cu. yards.

Life of Travelift Cables

By contrast the cables in the Travelift have short life. The endless conveying line takes four wraps around both the driven and gipsy drums which are under 4 ft. in diameter. This severe bending results in breakage as the sheave diameter is too small in relation to the cable diameter. The best type for this is the 6 × 19 improved plough steel prestressed flattened-strand cable with a wire core. The average life is unpredictable but may be around 250,000 cu. yards.

Fig. 5. Travelling tail tower at Hungry Horse dam

The endless transmission cable is the weakest link in the Travelift. This not only passes around the same diameter drive and gipsy drums as the endless conveying line but also through three transmission sheaves in the carriage with reversal of flexure there.

The change in tension in this cable during acceleration and deceleration of the load is more severe than that in the conveying line during acceleration and deceleration of the carriage. It is this transmission cable that breaks and drops the load. At Detroit dam in Oregon, which had two 8 cu. yard Travelifts, both lines parted within 24 hours of each other and dropped the load in the canyon. The bending and tension is so severe in this cable that the inner strands break as readily as the outer strands, making visual inspection of little value. The life of a transmission cable can be anything from 1,000 to 60,000 cu. yards. They have been lost in less than two shifts of operation. The splice is a rolling tuck splice some 120 ft. long. The cable naturally renders and must have the splice relocated at least once a day to keep it from getting on to the drums or hoist sheaves. The length of span determines how often it must be relocated, the object being to keep the splice between the head and tail towers on the direct return.

The weight of a Travelift carriage with hoist is 14 tons. This is all dead weight that must be hauled and supported. It requires more steel in the towers, bigger anchorage and larger guts. The dead weight on a conventional rig is 5½ tons.

The small diameters of the mainhoist drums and transmission sheaves are responsible for the excessive cable costs in a Travelift. The use of a wire rope for power transmission is highly unorthodox and impractical. Contact of the rope to the sheave groove for power transmission is not yet solved, and it is not unusual for the inserts to be wiped out in one shift. There are other mechanical troubles with a Travelift. There is heavy bending induced in the main hoist and gipsy shafts due to the eight parts of rope under 10 to 30 tons per part. At Lookout Point dam, nine 8½ inch diameter shafts were broken, and at Waneta dam, British Columbia, seven were broken. There is no torsion at the gipsy drum shafts as the drums are idlers. Walker rings or slip rings are used on both drive and gipsy drums, yet from an inspection of the broken shafts the failure has been due to torsion for the most part.

Layout

To cover the work, a cableway may be arranged to sweep either a rectangular area or a segment of a circle, depending upon whether both or only one of the anchorages can be moved laterally. The movement of the anchorages is usually achieved by a tower which travels on two tracks or by luffing towers operated by guys. Sometimes a travelling tower on a single track has been used in which the counterweight is balanced by the pull of the track cable. With this arrangement the sag is continually changing as the cableway is loaded and unloaded, and so makes the hook difficult to control.

It may be uneconomical to cover the whole work, and offset parts like the spillway channel may have to be concreted by alternative methods or by a separate cableway. The layout and elevation of the 8 cu. yard Donnells dam cableway is shown in Fig. 6.

If possible the cableway should be fed from the head-tower side, the tower having the hoist being termed the head tower and the other the tail tower. Also the anchorage on the pick-up side should be made higher than the other so that the power required may be reduced by reason of the load moving downhill.

The terrain often makes the provision of a track for the travelling towers quite costly. For instance at Table Rock dam, well over 100,000 cu. yards of fill were required for the track, and at Donnells dam some 40,000 cu. yards of rock had to be excavated from the precipitous side of the canyon to accommo-

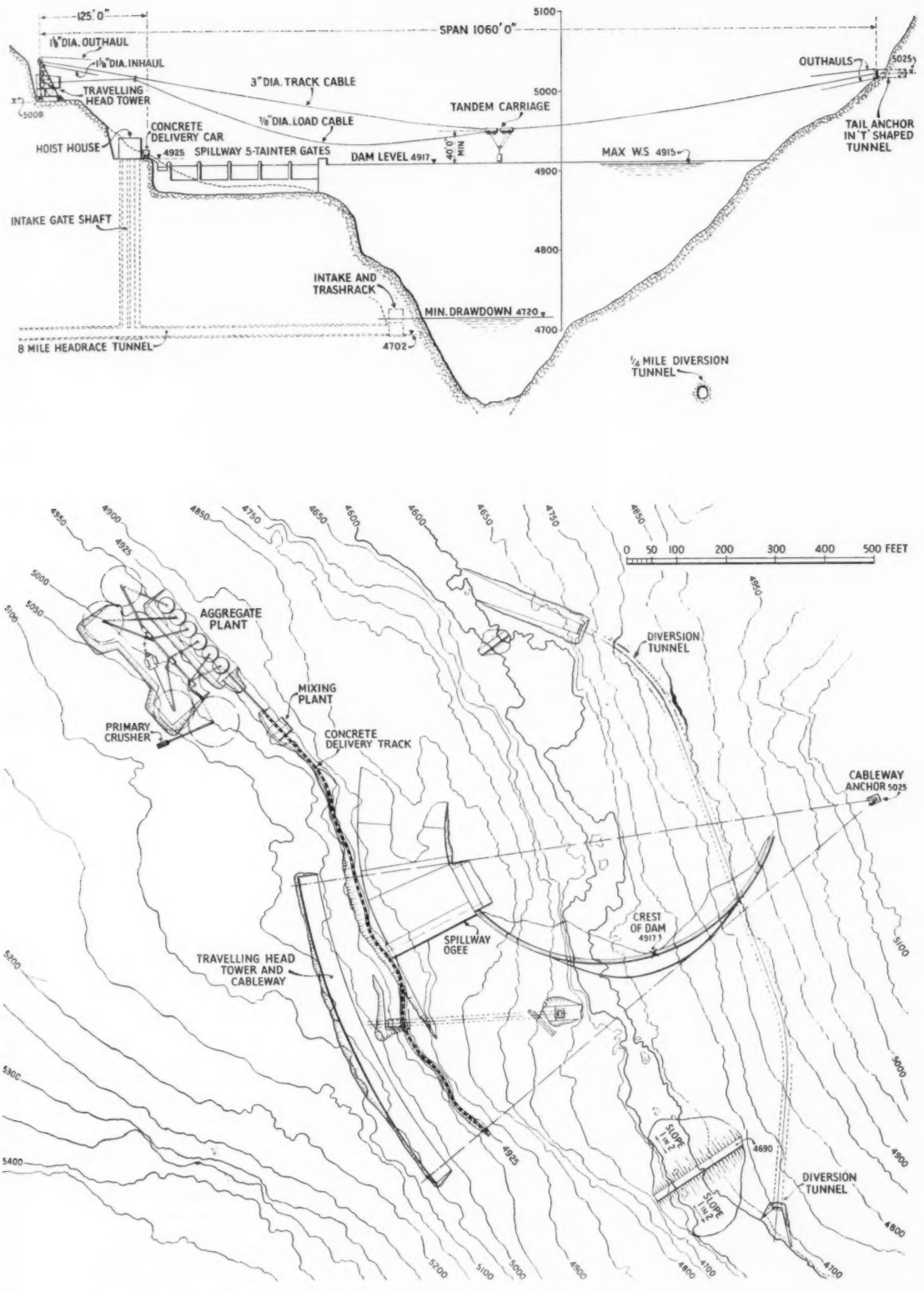

Fig. 6. Plan and elevation of Donnels dam and 8 cu. yard modified English-type cableway

date the travelling-head tower shown in Fig. 4.

The horizontal thrust of the travelling tower is normally supported by the front track, which is banked. The track must be very well built or differential settlement will cause interference to operation. Sometimes the horizontal thrust is carried by a double-flanged horizontal-wheel assembly running on a rail fixed to a reinforced concrete dwarf wall which has to be well anchored to the ground.

Donnells Dam Cableway

Typical of the American standard type cableway is the 25 ton Lidgerwood cableway used at Donnells dam in California by Morrison-Knudsen Company. This cableway had previously been used both at the Shasta and Hungry Horse dams. Whereas at these dams the span had been of the order of 2,000 ft., at Donnells dam it was only 1,100 ft. The cableway as used at Donnells uses the modified English system.

The travelling head tower is mounted on 40 wheels on 12 front and 8 rear axles. Front trucks consist of four 3-axle assemblies and rear trucks four 2-axle assemblies. Each leg of the tower is supported by an equaliser box girder spanning between a pair of trucks with ball and socket bearings. In the 3-axle trucks the central axle is set slightly higher than the other two axles to give an equal distribution under full load. The usual 4 ft. 8½ in. gauge track is used with 132 lb. per yard rail and timber sleepers. The two inside rear trucks each carry a drive mechanism consisting of a 75 h.p., 2,200 V a.c. motor, a gear reduction and a chain drive (total travel power 150 h.p.). The travelling tower, used as a tail tower, is shown at Hungry Horse dam in Fig. 5.

Fig. 7. Concrete placing at Donnells dam

It weighs 25 tons and is 52 ft. high from the track to main pin. It carries a counterweight 50% in excess of that required just to balance the maximum static pull of the track cable. At the top of the tower are two endless sheaves and a button line sheave (not in use).

The track cable take-up, located at the tail anchor, consists of a jewel (for connecting to the track cable by fanning out the separate strands into a cone-shaped mould and pouring in molten zinc); a 10 part 1⅛ in. cable take-up 25 ft. in length; and a pin connection to a universal joint which obviates bending in the track cable.

The carriage consists of two close-coupled units each carrying two load-line sheaves, and each unit is carried on the track cable by eight wheels in assemblies of two wheels each (16 wheels in all) designed to run on a 3 in. track cable. Although the carriage is provided for two 4-part ⅞ in. load lines, only one load line is in use at Donnells, and it is reeved in three parts. This is to increase speed of hoisting in the deep canyon. The button line and all except one slack carrier have been eliminated, advantage being taken of the fact that the span is less than that for which the cableway was originally designed. The weight of the fall block has been increased to hold down the empty hook. The load line runs out from the hoist in the travelling tower to the single slack carrier suspended from the track cable near the pick-up point. When the carriage is brought into the head tower for greasing, this slack carrier is withdrawn.

The hoist has one load-line drum (⅞ in. diameter cable) which is 6 ft. long and one endless driven drum (1⅛ in. diameter cable). For satisfactory working the first fixed sheave must be 110 ft. away from the drum. This is determined by the fleet angle which should not exceed 1½°—the fleet angle being the maximum departure from a right angle that the load line makes with the hoist drum. This is one of the factors that has to be taken into account in the design of a cableway and explains the arrangement at Donnells where the load line runs out to a single slack carrier. At Shasta dam a very high fixed head tower (common to seven cableways) was used, and at Hungry Horse dam the head tower was 195 ft. high (common to three cableways), so that in each case the fleet angle was small.

The hoist is powered by a 500 h.p. motor-generator

set, consisting of a synchronous 2,300 V a.c. motor connected to a d.c. generator producing up to 625 V. The drive is by a d.c. motor.

Height of Towers

The percentage sag k in a cable is given by $k=\frac{100}{8t}(WS+2G)$ where W is the weight of cable in lb. per ft., S is the span in ft., G is the concentrated load in lb., and t is the cable tension in lb.

By experience, the most favourable operating value for k is found to be between 5% and 6%. Taking into consideration the highest point in the work and the clearance needed for fall block and load, the height of the towers may be calculated. For a given sag the tension in the cable can be calculated from the relation above.

Operation

The operator should be in a position to have a good view of the job. At Table Rock dam the control cabin was mounted on a truck which travelled on the same track as the concrete delivery cars; but various locations are used. The operator must be in radio or telephone contact with a bell boy at the point of concrete placing. At Donnells dam, this placing point is shown in Fig. 7, where an air line is coupled up to the concrete bucket and provides the power to open the bucket gates. A recent development to eliminate the necessity for such a coupling up consists of a hydraulically-powered bucket, the power being stored up by the weight of the contents of the bucket when it is standing in the delivery car. The dump line, as shown in Fig. 3*c*, is another way of operating the bucket gates but has the objection that the actual operation of the gates becomes controlled by the cableway operator on telephone instructions and this is not altogether convenient in practice.

The controls of the cableway also include a device to show the cableway operator when his drums actually come to rest after he has applied the brakes. Television was used at Table Rock dam for this purpose.

The hoists and cables must have an oiler on duty at all times.

An 8 cu. yard cableway has a maximum output of 120 cu. yards per hour when travelling some 600 to 700 ft. at 1,350 ft. per min. This will require an 8 cu. yards capacity concrete plant to serve it. Average output of the cableway should be about 60% of the maximum capacity with the cableway carrying out such duties as moving reinforcing steel, forms and so on.

Acknowledgments

The author's thanks are due to Morrison-Knudsen Company, the contractors at Hungry Horse, Lookout Point, Waneta, Table Rock and Donnells dams for permission to reproduce from the *Em Kayan* Figs. 2, 3, 5 and 6, and to Mr. Roger C. Evans, Project Manager, Utah Construction Company, for supplying information on the Travelift cableway and on the life of cables. Mr. Evans was personally concerned with the erection or operation of all the cableways to which reference is made in the article.

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From page 341

earth dam would never be overtopped.* The severe flood which occurred in June, 1957, has revealed that there is an additional margin of safety in the resistance to erosion of the clayey fill.†

The upstream cofferdam, about 250 m. long and 13·5 m. high, had been completed at the end of April, 1957. It has a cross-section which is of the same type as the main dam, with a clayey core, of a mean thickness of approximately 4·5 m., supported by alluvial gravel and broken rock shoulders on both sides, the slopes being 2·5 : 1 upstream and 3 : 1 downstream. The clayey core had been deposited in 25 cm. layers, with a moisture content somewhat less than the optimum, and compacted by four passages of a 45 ton pneumatic roller with a tyre pressure of 90 lb. per sq. in.

On the 14th June, 1957, one diversion tunnel of 9·3 m. diameter and a capacity of 1,000 cu. m. per sec. was in use, and the upstream end of the second diversion tunnel was being finished behind a thin concrete arch acting as cofferdam. At midnight the discharge in the river was approximately 600 cu. m. per sec., and following torrential rainstorms the flow rose to a maximum of 1,700 cu. m. per sec. at 10 a.m., falling to approximately 900 by 6 a.m. on the 15th June. The level of the Durance upstream of Serre Ponçon rose rapidly until a sheet of water 2 m. thick was pouring over the upstream cofferdam. This overflow reached a maximum of approximately 400 cu. m. per sec. and lasted for nearly 20 hours. The area behind the upstream cofferdam was drained through an adit to the second diversion tunnel.

When the flood had subsided and it was possible to study the damage done to the cofferdam, the clayey core was found to have resisted the erosive forces, and also the hydrostatic load, without any serious slips, although much of its downstream support, consisting of gravelly layers, had been washed away. A complete repair was comparatively easy, speedy and cheap.

Acknowledgment. This article has been prepared by permission of M. R. Giguet, Deputy Director General of Electricité de France. M. Vaulpré, of their office at Gap, kindly assisted by providing some of the data. The photographs are by Mr. M. F. Clarke, a post-graduate student at Imperial College.

* See WATER POWER, Vol. 9, No. 5, May 1957, p. 181, and Vol. 9, No. 6, June 1957, p. 220.
† "La Crue de la Durance du 14 Juin 1957, à Serre Ponçon," *Construction*, Vol. 12, No. 12, December 1957.

Les Ateliers de Constructions Electriques de Charleroi have issued a handsome booklet with the idea of giving an illustrated synthesis of their manifold activities which include practically every aspect of modern engineering practice, from power plants to small-scale domestic appliances.

An Exponential Equation for Critical Depth in Circular Channels

By M. H. DISKIN, B.E., A.M.I.E.Aust.
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NOMENCLATURE

A = Cross sectional area of flow in channel.
B = Width of channel at the water surface.
D = Diameter of circular channel.
g = Acceleration due to gravity.
h = Depth of flow in channel.
h_c = Critical depth.
K = Numerical constant.
n = Numerical constant.
Q = Discharge.
Q_c = Critical discharge.

THE relationship between the discharge and the depth of flow in a channel when critical conditions prevail is unique, i.e. for each critical depth (h_c) there is one value of discharge which is the critical discharge (Q_c). This fact may be expressed as:—

$$Q_c = \phi\ (h_c) \qquad \ldots (1)$$

The functional relationship may be derived by differentiating the specific energy equation (assuming uniform velocity distribution and no curvature):—

$$e = h + \frac{Q^2}{2g\ A^2} \qquad \ldots (2)$$

with respect to the depth of flow (h) and equating the derived equation to zero for maximum discharge under given specific energy. The result obtained is:—

$$Q_c = g^{\frac{1}{2}}\ \frac{A^{3/2}}{B^{\frac{1}{2}}} \qquad \ldots (3)$$

Since both the area of flow A and the top width B are functions of the depth, the last equation defines the critical depth corresponding to the critical discharge Q_c.

For circular channels it is convenient to express the critical depth equation non-dimensionally, using the diameter of the channel (D) as denominator for length ratios:—

$$\frac{Q_c}{(g\ D^5)^{\frac{1}{2}}} = \frac{(A/D^2)^{3/2}}{(B/D)^{\frac{1}{2}}} \qquad \ldots (4)$$

The equation obtained is not explicit. Should an explicit equation be desired, equations relating the area ratio (A/D^2) and the top width ratio (B/D) to the depth ratio (h/D) must be substituted in equation (4) above. These equations may be derived from the geometry of the parts of a circle (Fig. 1):—

$$(A/D^2) = \tfrac{1}{4}\ \cos^{-1}\ (1 - 2h/D) - \tfrac{1}{2}\ (1 - 2h/D)\ (h/D - h^2/D^2)^{\frac{1}{2}} \qquad \ldots (5)$$

$$B/D = 2\ (h/D - h^2/D^2)^{\frac{1}{2}} \qquad \ldots (6)$$

Substitution of these equations in equation (4) will give a very cumbersome result, unsuitable for practical computations.

Numerical values of (A/D^2) and (B/D) for varying values of (h/D) are available in many handbooks. With the aid of a computing machine, numerical values of the expression $\frac{Q_c}{(g\ D^5)^{\frac{1}{2}}} = \frac{(A/D^2)^{3/2}}{(B/D)^{\frac{1}{2}}}$ were

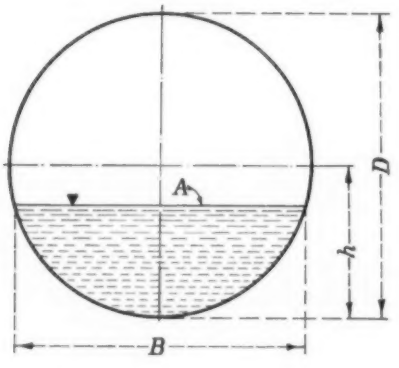

Fig. 1

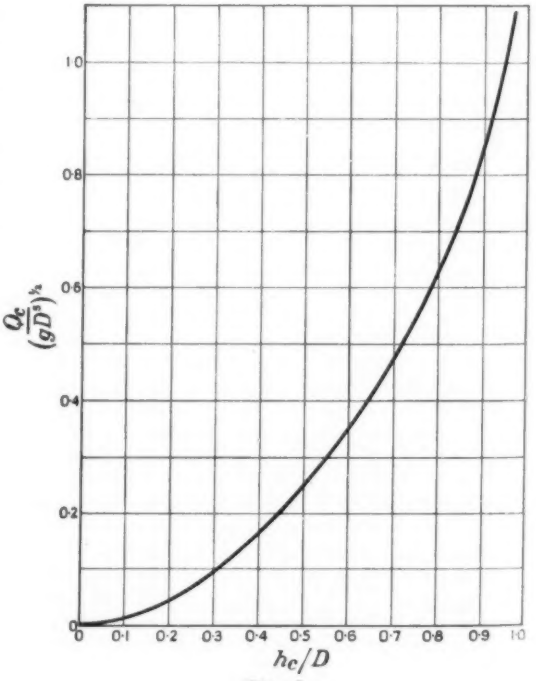

Fig. 2

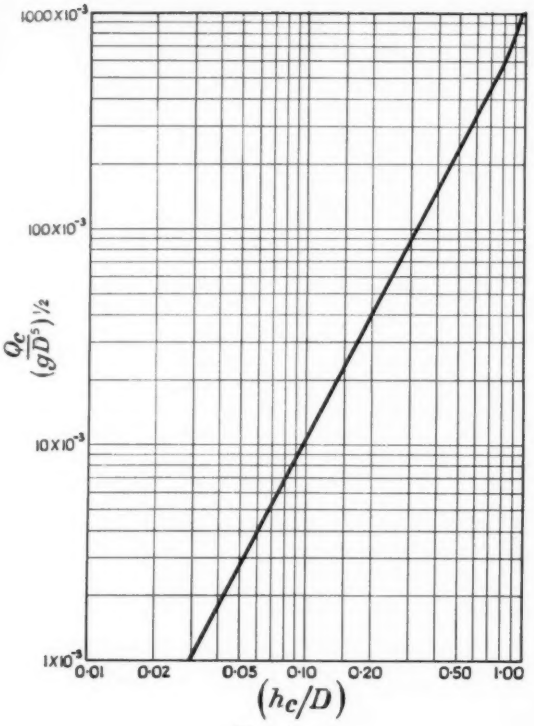

Fig. 3

computed (Table I) for varying values of (h_c/D) and plotted in Fig. 2. Plotting the same data on logarithmic paper (Fig. 3) gives a straight line which suggests a relationship of the type:—

$$\frac{Q_c}{(g\ D^5)^{\frac{1}{2}}} = K\left(\frac{h_c}{D}\right)^n \qquad \ldots (7)$$

or: $\log\ Q_c/(g\ D^5)^{\frac{1}{2}} = \log\ K + n\ \log\ (h_c/D)$ ... (8)

Least square curve fitting to the logarithms of values given in Table I in the range $0{\cdot}05 < h_c/D < 0{\cdot}85$ gave values of $n = 1{\cdot}95$, $K = 0{\cdot}954$.

TABLE I.—FLOWS FOR VARYING VALUES OF CRITICAL DEPTH IN CIRCULAR CHANNELS

h_c/D	A/D^2	B/D	$Q_c/(gD^5)^{\frac{1}{2}}$
0·00	0·0000	0·0000	0·000
0·05	0·0147	0·4359	$2{\cdot}699 \times 10^{-3}$
0·10	0·0409	0·6000	10·68
0·15	0·0739	0·7141	23·77
0·20	0·1118	0·8000	41·80
0·25	0·1535	0·8660	64·64
0·30	0·1982	0·9165	92·18
0·35	0·2450	0·9539	124·16
0·40	0·2934	0·9798	160·55
0·45	0·3428	0·9950	201·18
0·50	0·3927	1·0000	246·11
0·55	0·4426	0·9950	295·17
0·60	0·4920	0·9798	348·63
0·65	0·5404	0·9539	406·76
0·70	0·5872	0·9165	470·00
0·75	0·6318	0·8660	539·68
0·80	0·6736	0·8000	618·09
0·85	0·7115	0·7141	710·22
0·90	0·7445	0·6000	829·29
0·95	0·7707	0·4359	$1001{\cdot}8 \times 10^{-3}$
1·00	0·7854	0·0000	∞

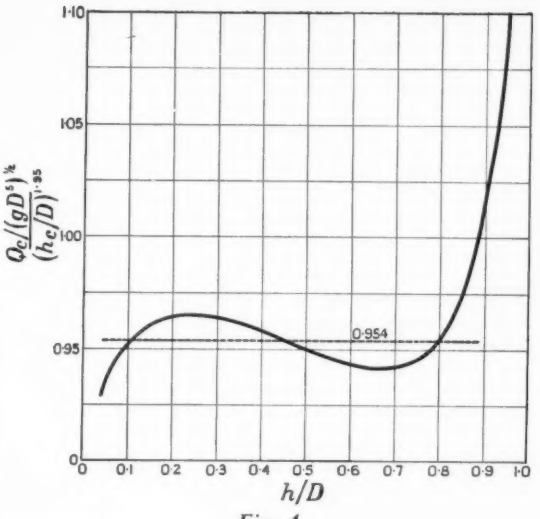

Fig. 4

The precision of the values of the constants was tested by assuming that the exponent $n = 1{\cdot}95$ is correct and computing the deviation of the quotient $\frac{Q_c/(g\ D^5)}{(h_c/D)^{1\cdot95}}$ from its mean value $K = 0{\cdot}954$. The mean deviation over the range of $0{\cdot}05 < h_c/D < 0{\cdot}85$ was 0·009, showing that the proposed critical depth equation

$$Q_c/(g\ D^5)^{\frac{1}{2}} = 0{\cdot}954\ (h_c/D)^{1\cdot95} \qquad \ldots (9)$$

gives results which are within 1% of those obtained from equations (4), (5) and (6).

A curve giving the values of the ratio $\frac{Q_c/(g\ D^5)^{\frac{1}{2}}}{(h_c/D)^{1\cdot95}}$ as a function of the depth ratio (h/D) is given in Fig. 4. This curve may be used for cases requiring better precision than that obtained by equation (9), although such precision is not justified in view of the assumptions contained in the specific energy equation (2).

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There will then apply for the approximating sine oscillation

$$s = A \sin 2\pi ft \qquad \ldots (7)$$
$$v = 2\pi fA \qquad \ldots (8)$$
$$a = 4\pi^2 f^2 A \qquad \ldots (9)$$

in which v indicates the maximum velocity of vibration and a the maximum acceleration.

An example of a vibration report is given in Fig. 8. The report should preferably only apply to one round. If it is a question of a delay, and not a short-delay round, the different delay numbers can immediately be distinguished and each is then indicated separately if the values are of interest. This is especially the case if a and γ assume high values ($\gamma = v/c$ in which c indicates the velocity of propagation). Even for a single charge several indications may be required because the vibration contains several different frequencies. Every one is then evaluated separately and is given a line in the report if the corresponding a and γ values are considered large. If c is not known v only is indicated.

(To be continued)

Economical Diameter for Penstocks

By G. S. SARKARIA, Senior Engineer, International Engineering Company, San Francisco, California

SELECTION of the most economical penstock diameter is one of the more important phases of hydro-electric design. The penstock should not only be capable of safely conveying the requisite quantity of water for the generation of desired power, but it should also do this efficiently and economically. Theoretically, there is only one penstock diameter which could be considered the most suitable for any particular installation. In actual practice, it may be desirable to have a penstock with a variable diameter, the average size being same as the economical diameter. This discussion is limited to uniform-diameter, welded-plate steel penstocks, although the procedures described will have wider application.

There are two methods of obtaining the economical diameter of a penstock. The first, an empirical approach, relies on simple formulae based on experience and well-tried data obtained from actual installations. If information regarding anticipated plant costs and future power requirements is not reliable, it is convenient to obtain the economical penstock diameter by using some of these empirical formulae. As shown by the example described in this paper, the author believes that the empirical formulae discussed herein give quite reliable results. The second, and the customary method, is to make detailed cost comparisons for different sizes of penstocks. Both these methods are described, and the results compared with the help of an example.

Empirical Formulae

A formula for economical penstock design, based on data from installations representing a very wide range of practice, and consistent with theoretical considerations, has been proposed by the writer[1]. It expresses the economical diameter as:—

$$D = 4{\cdot}44\frac{(P)^{0{\cdot}43}}{(H)^{0{\cdot}65}} \qquad \ldots (1)$$

in which

D = economical diameter of penstock in feet
P = rated horsepower of turbine
H = rated head of turbine in feet.

This formula is applicable primarily to power plants with Francis and propeller-type turbines, and gives reliable results for penstocks of 5 ft. or more in diameter.

Another empirical formula[2], also based on rated horsepower and head of turbine, is as follows:—

$$D = \left(\frac{P}{H}\right)^{0{\cdot}466} \qquad \ldots (2)$$

The U.S. Bureau of Reclamation[3] uses an empirical formula which expresses the economical velocity (ft. per sec.) as:—

$$V = 0{\cdot}125\sqrt{2gH} \qquad \ldots (3)$$

These three formulae are quite useful for obtaining quick results. To evaluate their usefulness, results obtained from them are compared with those obtained by lengthy cost analysis for the following example.

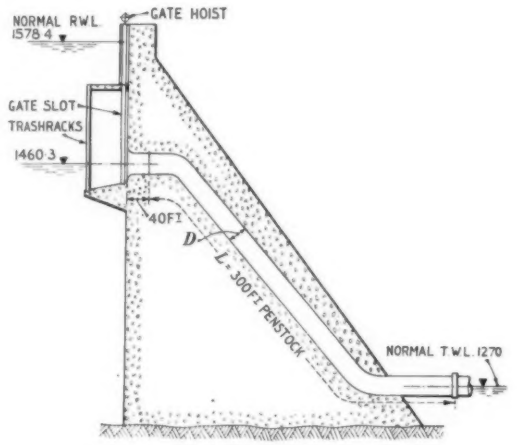

Fig. 1

Example: The data for the installation shown in Fig. 1 are as follow:—

Type of turbine	Francis
Rated head	$H = 280$ ft.
Rated horsepower	$P = 155{,}000$
Length of steel penstock	$L = 300$ ft.
Average allowable stress in penstock plate	15,000 lb. per sq. in.
Joint efficiency	90%
Load factor for power demand	60%
Assumed life of installation	50 years

It is desired to determine the economical penstock diameter.

Using the formula in equation (1), the economical diameter, $D = 4{\cdot}44\dfrac{(155{,}000)^{0{\cdot}43}}{(280)^{0{\cdot}65}} = 19{\cdot}4$ ft.

The formula in equation (2) gives the diameter as, $D = \left(\dfrac{155{,}000}{280}\right)^{0{\cdot}466} = 18{\cdot}8$ ft. Assuming economical velocity equal to $0{\cdot}125\ (\sqrt{2gH})$, or 16·8 ft. per sec., the penstock diameter should be 19·2 ft. To verify these results, six diameters from 17 ft. to 22 ft. at one foot intervals are examined and compared.

Average Annual Discharge

Since power production over a period of one year is unlikely to be uniform, it is desirable to base the size comparison on the estimated average annual discharge through the turbine. A power duration curve, which also indicates variations in required penstock discharge, is obtained from reservoir-operation studies

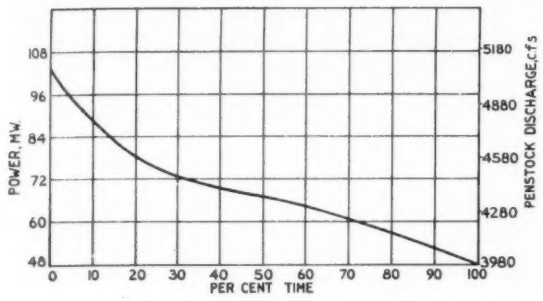

Fig. 2

based on available hydrological data. A typical power-duration curve, which is assumed to be applicable to the current example, is shown in Fig. 2. Values of discharge, Q, are plotted to such a scale that the power-duration curve also indicates penstock discharge duration. The average annual discharge is then equal to weighted mean value obtained from the discharge duration curve. From Fig. 2, the average annual flow through the penstock is computed to be 4,430 cusecs.

Shape of Intake

The intake is assumed to have a rectangular entrance, suitable for a fixed-wheel type gate, and a smooth gradual transition, 40 ft. long, from this rectangular shape to the circular penstock. In order to reduce losses at the intake to a minimum, it is estimated that area of the rectangular intake should be 1·67 times the area of the pipe. Also, the height of rectangular entrance is generally twice its width. The intake entrance dimensions and velocities are given in Table I for various trial diameters. However, it should be mentioned that the most suitable intake gate size is often governed by other considerations and criteria more decisively influential than the penstock diameter.

TABLE I

D (ft.)	Penstock area (ft.)^2	Intake area (ft.)^2	Intake size (ft.)	Q (c.f.s.)	Entrance velocity (f.p.s.)
17·0	228	380	13·7 × 27·4	2,658	7·0
18·0	255	424	14·5 × 29·0	2,658	6·3
19·0	284	473	15·4 × 30·8	2,658	5·6
20·0	314	524	16·2 × 32·4	2,658	5·1
21·0	346	577	17·0 × 34·0	2,658	4·6
22·0	380	634	17·8 × 35·6	2,658	4·2

The loss of head through the entrance (h_e) transition is assumed equal to $0{\cdot}2\frac{V_e^2}{2g}$. Later, when costs are estimated, the sizes of the gates and stop-logs are taken approximately equal to the intake sizes.

Head Losses

The following losses, except for velocity head h_v, are deducted from the gross head to obtain the net head:—

Trashrack loss, $h_{tr}=0{\cdot}50$ ft. (assumed constant) for a velocity of 2·0 ft. per sec.

Entrance loss, $h_e=0{\cdot}2\frac{V_e^2}{2g}=0{\cdot}2\ h_{ve}$

Velocity head, $h_v=\frac{V^2}{2g}$; V = average velocity in penstock

Transition loss, $h_t=0{\cdot}1\ (h_v-h_{ve})$

Bend losses, h_b

$=0{\cdot}15\ h_v$ for 35° bend at El. 1460·3

$=0{\cdot}19\ h_v$ for 52° bend at El. 1270·0

Friction loss, $h_f=0{\cdot}32\frac{(V)^{1{\cdot}9}}{(D)^{1{\cdot}1}}\frac{L}{1000}$

For $L=300$ ft., $h_f=0{\cdot}096\frac{(V)^{1{\cdot}9}}{(D)^{1{\cdot}1}}$

These losses for various trial diameters are given in Table II.

TABLE II

Head loss	Diameter of penstock (ft.)					
	17·0	18·0	19·0	20·0	21·0	22·0
h_{tr}	0·50	0·50	0·50	0·50	0·50	0·50
h_e	0·15	0·12	0·10	0·08	0·07	0·06
h_t	0·14	0·11	0·09	0·07	0·06	0·05
h_b (35°)	0·32	0·25	0·20	0·17	0·14	0·11
h_b (52°)	0·40	0·32	0·26	0·21	0·17	0·15
h_f	0·47	0·35	0·26	0·20	0·16	0·13
Total head loss, H_L =	1·98	1·65	1·41	1·23	1·10	1·00

Loss of Energy

The total head lost (H_L) in the penstock represents a direct loss of generating capacity. At a combined efficiency of turbine and generator of 85%, the annual loss in kW is:—

$$4{,}430 \times 62{\cdot}5 \times H_L \times 0{\cdot}746 \times 0{\cdot}85 \times \frac{1}{550} = 319 \times H_L \text{ kW} \quad \ldots (4)$$

The annual loss of revenue at 0·6 cents per kWh, and at 60 per cent. load factor, is equal to:—

$$319 \times \frac{0{\cdot}6}{100} \times 0{\cdot}6 \times 24 \times 365 \times H_L \text{ or} = \$10{,}000\ H_L \quad \ldots (5)$$

Estimation of financial loss on an annual basis only can be deceptive. As the loss is a recurring one, its capitalised value or present worth should be considered. Seeing that the estimated useful life of the installation is 50 years, and the assumed rate of interest as 4% (compound), the capitalised value of annual loss in dollars is $21{\cdot}482 \times 10{,}000\ H_L$. The capitalised values of power loss for different penstock diameters are given in Table III.

TABLE III

Penstock dia. (ft.)	H_L (ft.)	Annual power loss ($)	Capitalised value of power loss ($)
17·0	1·98	19,800	425,000
18·0	1·65	16,500	354,000
19·0	1·41	14,100	302,500
20·0	1·23	12,300	264,000
21·0	1·10	11,000	236,300
22·0	1·00	10,000	214,820

Cost of Penstock Liner

For estimating the cost of the plate-steel penstock, it is assumed that the liner is capable of withstanding internal hydrostatic as well as hydrodynamic pressure due to water-hammer. Pressure due to water-hammer is assumed to be a maximum at the downstream end of the penstock and equal to 30% of the gross head,

and it varies linearly to zero at the intake.

Gross head on penstock = 1578·4 − 1270 = 308·4 ft.

Total head at downstream end of penstock

$$H_{max.} = 1{\cdot}3 \times 308{\cdot}4 = 401 \text{ ft.}$$

Total head at upstream end of penstock

$$H_{min.} = 118{\cdot}1 + 13{\cdot}9 = 132 \text{ ft.}$$

The thickness of the plate for a pipe D ft. in diameter, subjected to a head h ft., for 90% joint efficiency, and 15,000 lb. per sq. in. allowable stress, is obtained from the thin cylinder formula as:—

$$t = 0{\cdot}0001929\ hD \text{ in.} \quad \ldots (6)$$

However, the thickness of plate should not be less than a certain minimum if the pipe is to be safe from buckling during erection. A formula commonly used to obtain minimum plate thickness is:—

$$\text{Minimum } t = \frac{12\ D + 20}{400} \text{ in.} \quad \ldots (7)$$

In Table IV, $t_{min.}$ is the plate thickness obtained from either equation (6) using $H_{min.}$, or from equation (7), whichever is greater. The maximum plate thickness is calculated by using $H_{max.}$ in equation (6). The average thickness, $t_{aver.}$, is used to determine the total weight of the 300 ft. long penstock from the following equation:—

$$\text{Weight of liner plate} = 38{,}600\ D\ t_{aver.} \text{ lb.} \quad \ldots (8)$$

TABLE IV

Dia.-D (ft.)	$t_{min.}$ (in.)	$t_{max.}$ (in.)	$t_{aver.}$ (in.)	Wt. of liner plate lb.
17·0	0·56	1·32	0·94	618,000
18·0	0·59	1·39	0·99	688,000
19·0	0·62	1·47	1·05	771,000
20·0	0·65	1·55	1·10	849,000
21·0	0·68	1·62	1·15	931,000
22·0	0·71	1·71	1·21	1,028,000

The cost of furnishing steel plate suitable for penstocks, and in various thicknesses, is estimated to be 15 cents per pound for all penstock sizes. The cost of fabrication and installing, including rolling, welding thermal stress relieving, X-ray inspection, and hydrostatic pressure testing is dependent on plate thickness as well as diameter of penstock. It is higher for thicker plates and larger diameters, and is estimated to vary from 12 to 17 cents per pound as indicated in Table V.

TABLE V

Dia.-D (ft.)	Wt. of liner (lb.)	Cost of plate ($ per lb.)	Fabrication and installation ($ per lb.)	Total cost of liner ($)
17·0	618,000	0·15	0·12	166,800
18·0	688,000	0·15	0·13	192,500
19·0	771,000	0·15	0·14	224,000
20·0	849,000	0·15	0·15	254,500
21·0	931,000	0·15	0·16	289,000
22·0	1,028,000	0·15	0·17	330,000

Reinforcement Around Penstock

When a penstock is embedded in the body of the dam, as shown in Fig. 1, a considerable amount of reinforcement is provided around the liner. The purpose of this reinforcement is to prevent cracking in the concrete surrounding the liner, due to stress concentrations caused by stresses in the dam and when the penstock is empty. Design of this reinforcement is complicated and lengthy, and is therefore not included in this article. For a 20 ft. diameter penstock and size of dam shown in Fig. 1, the circumferential reinforcement was estimated to vary between one row of 1¼ in. diameter bars at 12 in. spacing around upstream end of liner, to two rows of 1½ in. diameter bars at 6 in. spacing around downstream end of penstock. In addition, longitudinal bars, ¾ in. diameter at 12 in. spacing, are provided as distribution reinforcement. The total quantity of reinforcement is dependent on diameter of penstock, and is estimated in Table VI for different sizes. The price of reinforcing steel bars is assumed as 10 cents per pound.

TABLE VI

Dia.-D (ft.)	Reinforcement around liner (lb.)	Cost of reinforcement ($)
17·0	300,000	30,000
18·0	315,000	31,500
19·0	330,000	33,000
20·0	350,000	35,000
21·0	365,000	36,500
22·0	380,000	38,000

These estimates do not include reinforcement around the unlined intake transition and in the trashrack structures. To make the comparison less involved, this reinforcement is assumed to be equal for all penstock sizes. This quantity is estimated to be about 100,000 lb., and its cost is later added to the total cost of each penstock.

Erection Supports

Structural steel saddles are commonly used to support sections of inclined penstocks during installation. These supports are embedded in the concrete as the dam rises, and therefore should be included in the penstock cost estimates. It is assumed that penstocks will be partly fabricated in the workshop, and pipe sections 30 ft. long will be installed at one time. The weight of a typical support for a 30 ft. length of 20 ft. diameter penstock is estimated to be 4,200 lb. Since ten such supports would be required, the total weight of steel erection supports for a 20 ft. diameter penstock would be 42,000 lb. For other sizes, the quantities of steel supports required, and their cost at 20 cents per lb., are given in Table VII.

Intake Gate, Hoists, Trashracks

As mentioned earlier, the size of the rectangular intake entrance depends on the diameter of the pen-

TABLE VII

Dia.-D (ft.)	Weight of erection supports (lb.)	Cost of erection supports ($)
17·0	40,500	8,100
18·0	41,000	8,200
19·0	41,500	8,300
20·0	42,000	8,400
21·0	42,900	8,580
22·0	44,100	8,820

stock. The larger the penstock diameter, the larger will be the intake, and therefore the larger the size of gate required. These gates are designed for a head of 118 ft., and their sizes and cost are estimated for various penstock diameters in Table VIII. The cost of furnishing and installing fixed wheel gates is taken at 75 cents per lb.

TABLE VIII

Dia.-*D* (ft.)	Size of gate (ft.) Width	Height	Wt. of gate (lb.)	Cost of gate ($)
17·0	14·0	28·5	92,000	69,000
18·0	15·0	30·0	105,000	78,800
19·0	16·0	32·0	122,000	91,300
20·0	16·8	33·5	130,000	97,500
21·0	17·6	35·2	150,000	112,500
22·0	18·5	37·0	175,000	131,200

Gate guides for various gate sizes do not vary appreciably as long as the head for which the gate is designed does not change. Estimated weights and cost of embedded metal work for gate guides are given in Table IX.

TABLE IX

Penstock Dia.-*D* (ft.)	Wt. of embedded metal guides for gate (lb.)	Cost ($)
17·0	36,600	12,800
18·0	36,900	12,900
19·0	37,200	13,000
20·0	37,600	13,150
21·0	38,000	13,300
22·0	38,400	13,420

If an individual hoist is provided for each penstock gate, the entire cost of the hoist should be charged to the cost of the penstock. The size of the hoist increases with the size of the gate, but the cost does not, however, increase in direct proportion to the lifting capacity. The cost of cable hoists for various penstock sizes are estimated as in Table X.

The cost of trashracks should also be debited to the penstocks. However, it is assumed that the same trashracks are suitable for all the penstocks under consideration. Their weight is estimated to be 130,000 lb. and the cost to be $30,000.

Total Comparative Cost

The total cost of penstock and auxiliary items as used for economic comparison is the sum of costs of the steel liner, reinforcement steel, erection supports, gate, gate guides, hoist, and trashracks. These are summarised in Table XI for various penstock sizes.

TABLE X

Penstock Dia.-*D* (ft.)	Cost of gate hoist ($)
17·0	17,000
18·0	18,200
19·0	19,500
20·0	20,200
21·0	21,100
22·0	22,000

Economical Diameter

To obtain the comparative cost of each alternative, the capitalised value of the power loss is added to the sum of direct costs. These totals are plotted in Fig. 3. The capitalised cost of power loss is also plotted.

The curve for total cost flattens from 18 ft. to 20 ft. penstock size, with a minimum at about 19·2 ft. which, therefore, should be considered the most economical penstock diameter for this installation. The curve for cost of lost power continuously flattens as the penstock diameter increases, indicating that the reduction in the power loss by increasing the penstock diameter beyond 21 ft. would not be substantial.

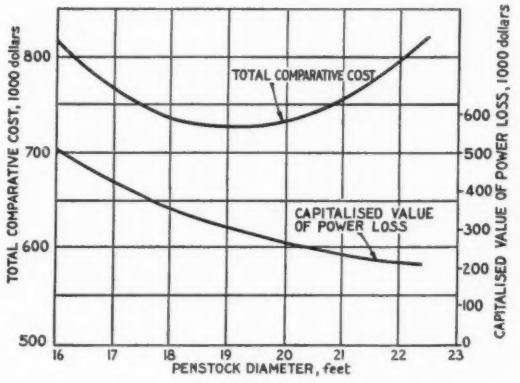

Fig. 3

The economical penstock diameter arrived at by this detailed and lengthy analysis checks very closely with those obtained by using three different empirical formulae. This agreement testifies to the usefulness of empirical formulae based either on data from existing installations or on experience. The cost analysis, however, is also dependent on a number of variables that have to be carefully selected, and their values should be realistic and representative of actual conditions prevalent or expected during construction and operation of the plant.

When the data selected for the various components truly reflect the actual conditions the detailed analysis is correct and well worth while. In cases where such factors as the cost of installing different

TABLE XI.—COST IN $1,000, PENSTOCK DIAMETERS IN FT.

	17	18	19	20	21	22
1. Steel liner	166·8	192·5	224·0	254·5	289·0	330·0
2. Reinforcement	40·0	41·5	43·0	45·0	46·5	48·0
3. Erection supports	8·1	8·2	8·3	8·4	8·6	8·8
4. Intake gate	69·0	78·8	91·3	97·5	112·5	131·2
5. Gate guides	12·8	12·9	13·0	13·2	13·3	13·4
6. Gate hoist	17·0	18·2	19·5	20·2	21·1	22·0
7. Trashracks	30·0	30·0	30·0	30·0	30·0	30·0
Total, direct cost	343·7	382·1	429·1	468·8	521·0	583·4
Capitalised value of power loss ...	425·0	354·0	302·5	264·0	236·3	214·8
Total comparative cost	768·7	736·1	731·6	732·8	757·3	798·2

types of equipment, the load factor, or the expected life of the installation, are not considered to be reasonably exact, the designer may be tempted to reject results of detailed cost analyses, and rely on proven empirical formulae. The penstock size thus determined would not be too far removed from the true economical diameter.

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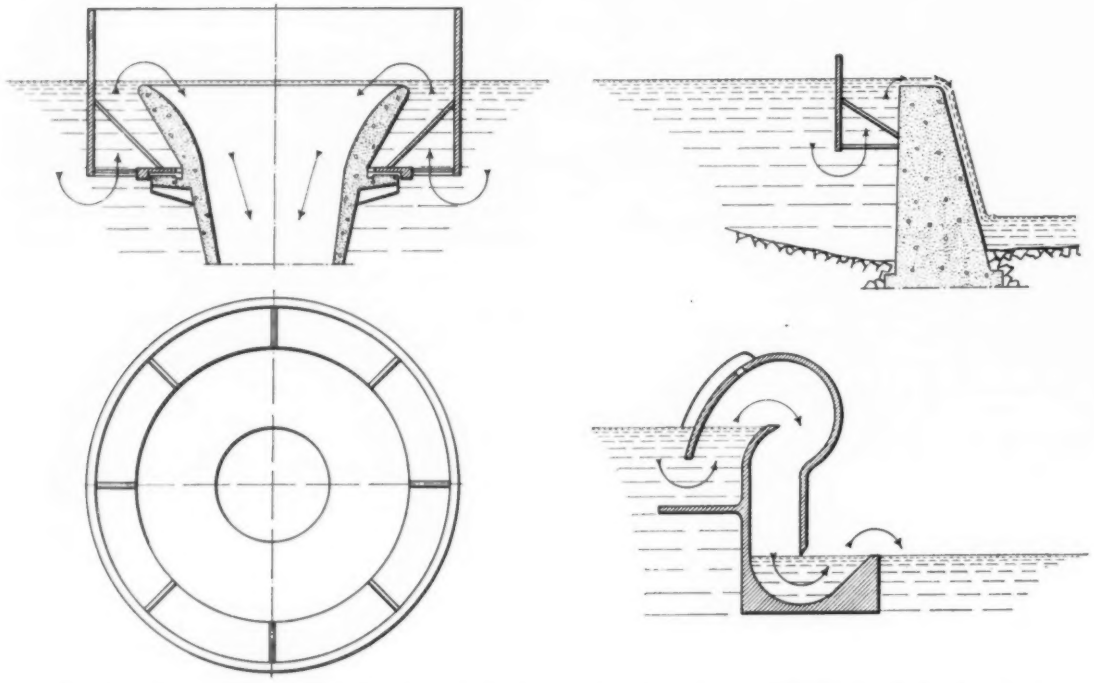
[3] "Welded Steel Penstocks Design and Construction," by P. J. Bier, *Engineering Monograph No. 3*, U.S. Bureau of Reclamation. July 1949.

Prevention of Evaporation from Reservoirs



Sketches showing overflow barriers to prevent loss of oil film, as suggested by Prof. Tortolina

The accompanying sketches have been sent to us by Prof. Dott. Ing. Renzo Tortolina, of Padua, who has taken out an international patent to protect his idea which consists simply of floating a thin layer of naphtha or some other non-volatile mineral oil on the surface of the water and so preventing any evaporation from taking place. An oil layer of 1 mm. in thickness is stated to prevent the evaporation of the underlying water, even in such latitudes as the Sudan where the tropical sun is estimated to evaporate water surfaces to depth of 5 metres per annum. The sketches refer to the special overflow arrangement that would be necessary to prevent the oil layer from being carried away by floods. The morning-glory spillway on the left consists of a normal well-type overflow around which a vertical enclosure of sheet metal has been erected to cause the spill to be drawn off from well underneath the surface, thus preventing the oil layer from flowing away. The arrangements shown in the two sketches on the right have the same objective. All these devices will, of course, be dimensioned in accordance with the overflow requirements.

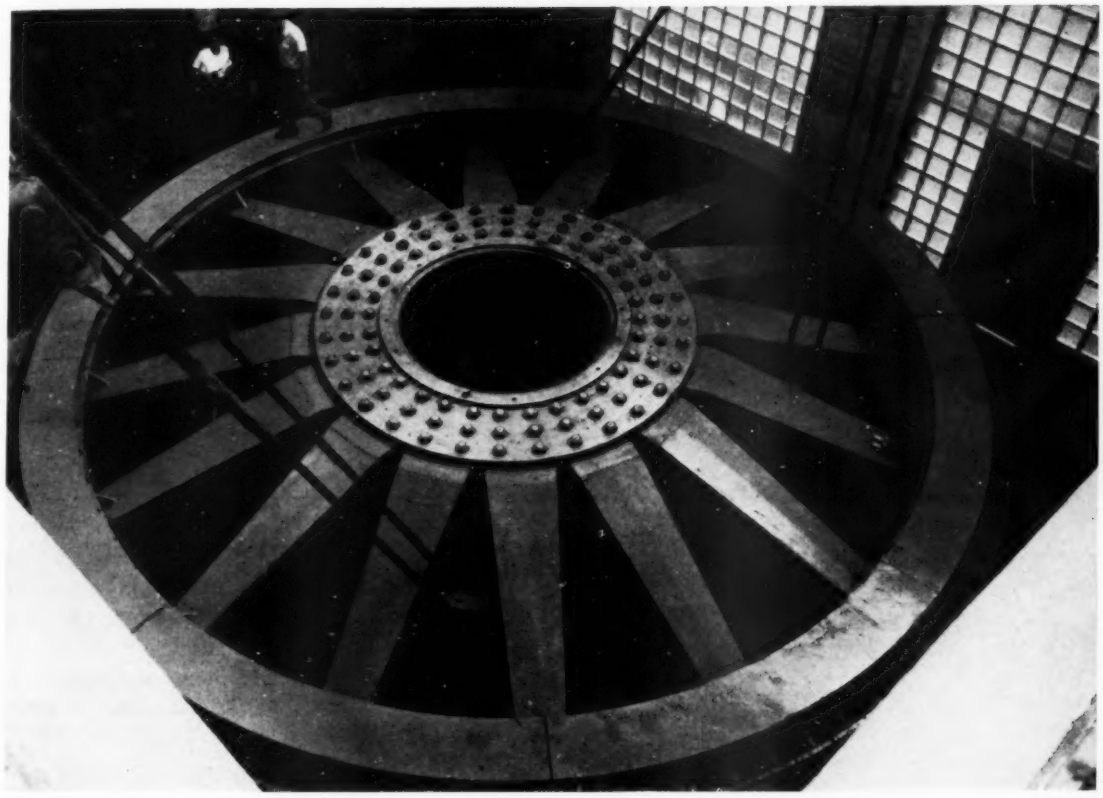
It is suggested that fish life could be sustained by injecting compressed air in depth so that the water could be maintained at a normal state of aeration, but no comments are offered as to how the oil film would affect insect life on which so many fish feed.

A transparent oil having a specific gravity of 0·88 is recommended so that the surface of the lake or reservoir would not be darkened.

Prof. Tortolina reports that in an accelerated laboratory test, under identical conditions, an unprotected sample of water lowered 35 mm. in 30 hours, whereas the surface of a sample protected by a layer of naphtha remained unaltered.

The International Electrotechnical Commission, which is affiliated with the International Organisation for Standardisation, has issued "Recommendations for Lightning Arresters," Part I—Non Linear Resistor Type, Publication 99-1, obtainable from the Bureau Central de la Commission Electrotechniques Internationale, 1, rue de Varembé, Genève, Suisse. Price Sw. Fr. 8.

First Rotor for Ybbs-Persenbeug North



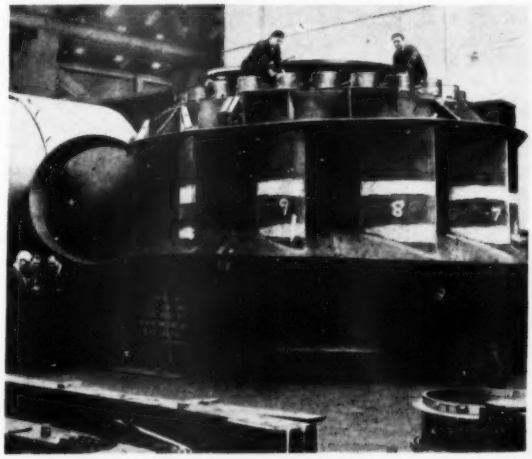
ELIN A.G. have delivered the first rotor for the northern section of the Ybbs-Persenbeug power station on the Austrian Danube (WATER POWER, June 1956). Due to transport problems it was found necessary to deliver this rotor in sections and some six weeks were required for final assembly in the power station. The unit weighs 230 tons and construction of the complete generator took 72,000 work-hours. Both the 130 ton overhead cranes had to be installed before the giant rotor could be placed in position.

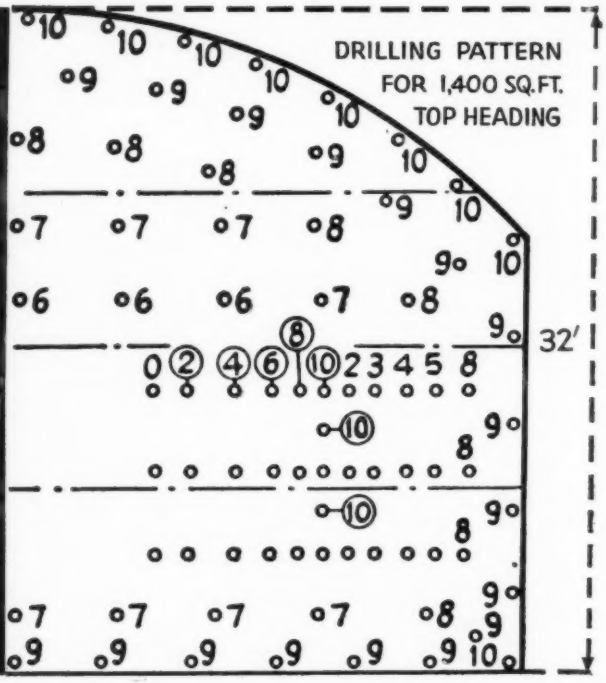
The Kaplan turbines, operating under a net head varying between 6 m. and 14 m., will have a capacity of 34,400 kW at a net head of 12·45 m. and a discharge of 310 cumecs. The runners are 7·4 m. in diameter and weigh 754 tons.

The northern section is expected to deliver power to the Austrian grid for the first time in early October 1958. The southern section has produced some 455 million kWh since the first two units began operating last autumn.

Kariba Turbines

The major parts of the first 140,000 h.p. Francis turbine for the Kariba hydro-electric power station have recently been despatched to Rhodesia ahead of schedule. Six turbines for this power station, with a total output of 840,000 h.p., are being supplied and erected by Boving & Co. Ltd., London, and manufactured at the works of Markham & Co. Ltd., Chesterfield. The recent shipments, weighing over 270 tons, comprise the stayring, spiral casing and guide apparatus but exclude the rotating parts of the tur-





Harrsele—a remarkable tunnelling operation in Sweden

Since the tunnelling operations began at Harrsele in northern Sweden the fast progress made there has been followed with great interest by construction and mining engineers throughout the world. On the 1,400 sq. ft. (32 ft. high, 49 ft. wide) top heading of the two-mile long discharge tunnel (the remaining bench to be excavated amounted to another 1,400 sq. ft.), the tunnelling crews beat the world tunnelling record in August 1955 with an advance of 585 feet, achieved by rounds of 26.4 feet. The work was carried on in three shifts; one for the drilling and blasting, and two for the mucking-out. On December 21, 1955, the break-through took place *—three months ahead of schedule.*

The 'Swedish Method' again!

The rock drills chosen for the Harrsele tunnel were exclusively Atlas Copco. Most of these were fitted with Sandvik Coromant steels. These lightweight drills and integral steels represent the basis of the 'Swedish Method'. To begin with, both were developed to work together so that drill and steel gave a completely balanced performance. Atlas Copco drills are small and compact. This means more drills at the face and a heavier concentration of drilling. The Sandvik Coromant steels with their new revolutionary SR anti-corrosion treatment have up to 50% longer life than untreated steels. The following drilling figures from the drifting at Harrsele show the impressive results to be gained from the 'Swedish Method'.

Advance per round	**26.4 feet**
Number of cu. yds. per round	**1,375**
Number of holes per round	**150**
Footage drilled per round	**3,940 feet**
Footage drilled per cu. yd.	**2.85 feet**
Number of drills and drillers	**21**
Number of man-hours per cu. yd. *(drilling, charging and blasting)*	**0.122**
Drilled footage per man-hour	**53.5 feet**
Advance per month *(27 working days)*	**580 feet**

Atlas Copco AR compressors supplied the air

The rock drills were fed from a battery of Atlas Copco AR 4E compressors. The famous AR-series give more air to the h.p. than other machines of their class.

World-wide sales and service

The Atlas Copco Group puts compressed air to work for the world. It is the largest group of companies specialising solely in the development and manufacture of compressed air equipment. It embraces Atlas Copco companies or agents manufacturing or selling and servicing Atlas Copco equipment in ninety countries throughout the world. For further details of the equipment featured here, contact your local Atlas Copco company or agent, or write to Atlas Copco AB, Stockholm 1, Sweden.

Atlas Copco *Manufacturers of Stationary and Portable Compressors, Rock-Drilling Equipment, Loaders, Pneumatic Tools and Paint-Spraying Equipment*

bine which are not required until a later stage of the erection programme. The first draft tube liner has already been assembled at site ahead of programme by Boving's erection staff and it is hoped to begin installation of the remainder of the turoine early in August when the concreting of the foundations will have been completed.

In the meantime, work is proceeding on the erection of the draft tube liners. The first generating set is scheduled for commissioning by 1st January, 1960.

New Electronic Equipment

Winston Electronics Limited, of Shepperton, Middlesex, have been appointed United Kingdom distributors for Beckman/Berkeley Industrial Electronics Instruments. This equipment will be manufactured by Beckman Instruments Limited, the British subsidiary of Beckman Instruments Inc., Richmond, California, U.S.A., in a new ultra-modern factory at Glenrothes, Scotland. This factory, which was officially opened on May 1, 1958, has under-floor heating and 80 candle lighting. Many of the Beckman/Berkeley instruments will have a direct application in power station operation and maintenance; the range includes Universal Eput meters, portable Universal counters, digital recorders, preset counter-controllers, high-speed electronic counters, helipots, etc. A. Gallenkamp and Co. Ltd. have been appointed to undertake the marketing of Beckman/Berkeley process control instruments.

Heavy-Duty Shovels

Among the important export orders placed with Ruston-Bucyrus Limited during June was one 110-RB Ward-Leonard electric quarry shovel for Carrières et Fours à Chaux de la Meuse, Belgium. The machine has a working weight of 152 tons and a dipper capacity of 4½ cu. yards. Other orders received were two 38-RB 1½ cu. yard and one 22-RB ¾ cu. yard excavators for the Maldive Islands and four 22-RBs for Fiji.

Visitors from abroad included engineers from Yugoslavia interested in heavy duty shovels for mining operations who paid visits to various opencast coal sites and to see especially the recently introduced 150-RB Ruston-Bucyrus 6 cu. yard shovel in operation at the Oxbow site of the well-known contractors, Sir Lindsay Parkinson & Son Limited. The 150-RB, shown in the maker's test yard, has an operating weight of 195 tons.

Tests on High-Pressure Branch Pipes

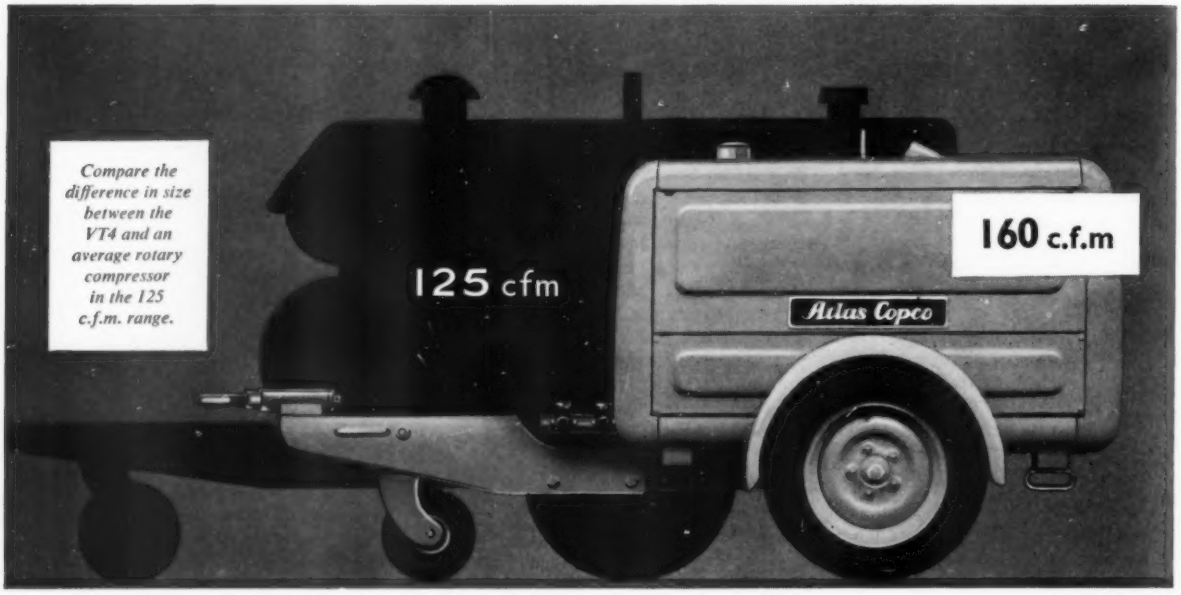
Distributors and branch pipes play a particularly important role in the high-pressure pipe lines of hydro-electric power stations. Located for the most part within the stations themselves, they are highly

stressed components in which flow conditions and pressure losses are more critical than at any other point along the whole pipe line. The weakening of the structure where the branches are joined to the main pipe calls for special designs based on careful preliminary calculations. The collar stiffening system introduced by Sulzer Brothers for branch pipes and bifurcations has been further improved in recent years, both in hydraulic and in mechanical respects, by the application of flow-research and systematic stress measurements. Better collar designs have been developed to facilitate the fabrication and transport of bulky, thick-walled branch pipes and to permit assembled pipe structures to be heat-treated and pressure-tested as a whole.

The loosely fitted reinforcing collar, which is not

NO OTHER COMPRESSOR

—piston or rotary—delivers so much air for its size and weight


Compare the difference in size between the VT4 and an average rotary compressor in the 125 c.f.m. range.
125 cfm
160 c.f.m
Atlas Copco

PACKAGED POWER IN THE ATLAS COPCO VT4— THE WORLD'S LIGHTEST 160 c.f.m. COMPRESSOR

The VT4 is over 20% lighter and 25% smaller per c.f.m than any machine, piston or rotary, in its range.

Lighter and smaller than some machines with a free air delivery of only 60 c.f.m.—the VT4 has taken power weight ratios into a new dimension.

Powered by a Deutz diesel engine the VT4 gives an unequalled c.f.m./h.p. ratio, with fuel economies which offer a saving on any contract.

The VT4 is the first of a series based on major changes in design and production methods. For example, it is the only machine with a single welded unit, which incorporates the crankcase, bellhousing and cylinder ducting.

Basic simplicity in design eliminates maintenance and overhaul problems—local mechanics anywhere can service this machine.

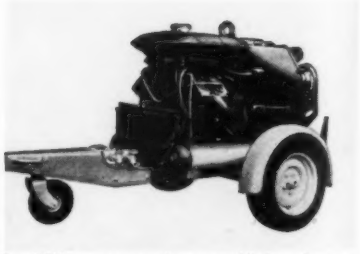
Unscrew one nut and all four valves in the low pressure cylinder can be removed. Loosen three bolts and the complete engine-compressor unit lifts out.

The VT4 is 100% air-cooled—starts on the button and works with equal efficiency anywhere in the world.

The VT4 is easier to load with its low weight and built-in lifting eye—easier to tow with its two wheels and torsion bar suspension.

Manhandling on site is a simple operation with a machine weighing less than 21 cwt. and fitted with a pivoting nose wheel.

All instruments and controls are operated from a central dashboard.



The VT4 is compactly assembled under a canopy which can be completely stripped in a few minutes to give maximum access.

THE *NEW* VT4

Atlas Copco PUTS COMPRESSED AIR TO WORK FOR THE WORLD

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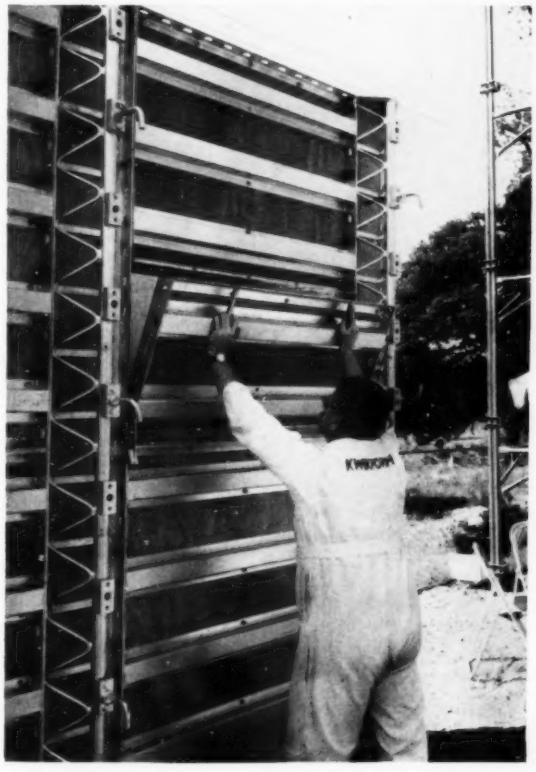
P.26

welded to the pipes, allows unobstructed changes of shape to be made in both the pipe and collar and thus prevents undesirable compressive stresses. As a result, stress conditions and safety factors are considerably improved.

The accompanying illustration shows a Sulzer branch pipe for a large pumped storage station undergoing a pressure test and stress measurements. The pipe diameters are from 80¾ to 32¼ in., the wall thicknesses from 2⅜ to 1¾ in. and the test pressure is 2,050 lb. per sq. in.

Plywood Formwork

A new type of plywood formwork has been designed by Kwikform Limited of Waterloo Road, Birmingham. This system consists of 4 ft. wide plywood sections with heights ranging from 5 ft. to 4 in., so that any dimension to the nearest 4 in. in height can be achieved. The aligners are 1 ft. wide and 10 ft. long and they are normally fitted in pairs 4 ft. apart. A special transfer aligner has been designed so that the plywood forms can be used in conjunction with standard steel sections. A 4 × 5 ft. hinged panel is provided so that concrete can be poured at a conveniently low level.



Plywood formwork is quick and easy to erect, reduces labour costs and provides an unusually good finish.

Draining Compacted Concrete

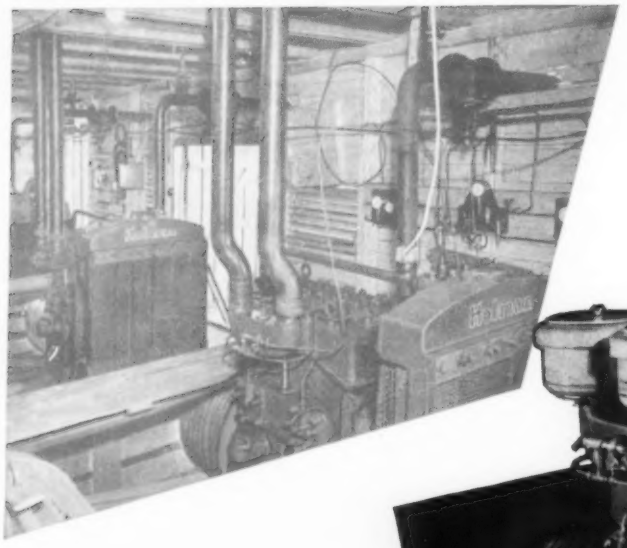
After trying syphoning, compressed-air pumps, and various other methods, the engineers on the Grande Dixence project in Switzerland ordered a group of

submersibles for pumping off surplus water exuded during consolidation of the concrete. The new pumps were put to work this spring. The concrete for the blocks of the dam is hoisted to the working level in loads of 8¼ cu. yards and is spread by bulldozers. Five vibrators mounted on a bulldozer are used for compaction. The Grande Dixence dam will be the highest in the world, 930 ft., and some 2,300 ft. in length. The enormous quantities of concrete being poured make the pumping off of the water from compaction a sizable job in itself.

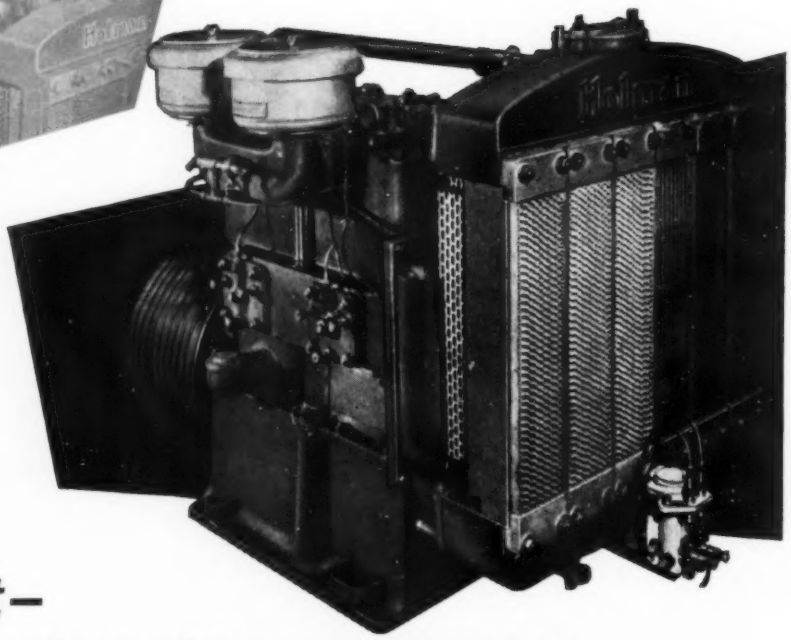
The pumps chosen, after a month's trial on the job, are Flygt B-38L's, manufactured by the Stenberg Corporation AB, of Stockholm, Sweden. The B-38L is a 1½ in. pump with a capacity of 80 g.p.m. at a head of about 20 ft. It can be moved around by one man—a considerable advantage—and it was also found during the trial period that this pump could handle the thick, concrete-loaded water without clogging. The compressed-air pumps previously used were stated to be very expensive because of the repairs required.

The Flygt range includes five electric submersible pumps in sizes from 1½ in. to 8 in.

Water-Power Development in Japan. A publication received from the Japan Dam Association reviews the work that has been done in regard to hydro-power development since the war. At the end of hostilities Japan was confronted with an acute shortage of power and it was decided that a special corporation (The Electric Power Development Co. Ltd.) should be established to work out a programme and arrange for the finance. Since its inauguration, this company has been responsible for completing seven projects having a total installed capacity of 519,700 kW. The largest is the Sakuma plant with an installed capacity of 350,000 kW. Nineteen more projects are in course of development and when these are completed the total



F.A.D. 525 cu. ft. min.
at 100 lb. per sq. in.



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Two-staging, a feature possessed by all Holman stationary compressors, does more. It gives even torque, economical lubrication, less strain and wear, longer working life.
Cooling is by a closed-circuit system, pressurised to 4 lb./sq. in., which makes evaporation losses negligible — a major consideration when a constant water supply is doubtful — and obviates external pipework.
The Holman T60R, like all Holman equipment, is built to *last*. And it's designed to cut down running cost — the *real* cost of compressed air equipment.
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hydro-electric generating capacity will amount to 1·5 million kW.

The publication is handsomely produced and bears the somewhat equivocal title of "Japan's Dam Skill at World's Top Level." Brief details of each scheme are given and the illustrations are both copious and excellent.

Abstracts from the World Technical Press

A Novel Dam-Building Technique

The dam-building procedure described in this article has been evolved from the technique applied by Soviet and Chinese engineers to construct the piers for the Wuchang bridge on the Yangtsé River. The depth of the river at the bridge site can be up to 35 m. during the flood period which may extend from June to October. Under such conditions, the use of caissons is, of course, impossible, and the method consists essentially in sinking a nest of precast ferro-concrete pipes into the river bed, and upon these the bridge-pier is built. Construction work began by lowering a large cylindrical steel carcase on to the river bed by means of floating cranes. This structure is very much like a wheel divided into radial compartments through which the pipes are inserted at the rate of 30 to 35 per carcase. The pipes, 155 cm. in external diameter and 10 cm. in wall thickness, are assembled in section of 3, 6, 9 and 12 m., according to requirements. They are then sunk through a layer of marl, 5 to 7 m. thick, down to the bed rock by means of pile-drivers. Their lower ends are packed with concrete plugs. Once a pipe is firmly in position, it acts as a guide to boring gear fitted with a 130 cm. diameter drill which drives through the solid rock to the required depth. This hole is reinforced by a steel basket which extends upwards into the reinforced concrete pipe; the pipe and hole are then filled with concrete. Upon completion of this procedure for all the pipes, a protective shell of sheet-piles is built around the carcase. The intermediary spaces around the pipes are also filled with concrete, and the top of the carcase, together with the top ends of the pipes, is covered by a concrete platform from which the construction of the pier proceeds by normal methods. The authors discuss the possibility of applying this method to the building of rockfill and concrete gravity dams in running water. In rockfill dams, the upstream slope would be interrupted by an external cut-off wall consisting of a row of vertical piers, set up at 25 to 30 m. apart. These buttresses, built on the pattern of the piers of the Yangtsé bridge, would be constructed with three vertical grooves on either side, two to accommodate either sheet piling or stoplogs, and the third, at the back, screens. Rubble would be dumped behind the screen across the whole river bed and the rockfill would proceed normally as soon as the closure of the river bed was completed. The height of the piers or buttresses in rockfill dams, and of the coffer-dam blocks in concrete structures, must obviously be calculated so as to ensure that their top edges still emerge from the water when the closure of the river bed is completed. Although six different solutions are described, there is no mention in the article that any project of this type is under consideration at the present time and, taking into account the importance and complexity of the problems involved, it is obvious that extensive preliminary studies and field investigations will have to be carried out before the technique used for the Yangtsé bridge-piers can be applied to major hydro structures. (I. N. Varonkin and V. N. Rudakof, *Gidrotechnicheskoe Ctroitelsvo*, Vol. 27, No. 2, February 1958, p. 6, 6 pp., 7 ff.)

Mauvoisin Development

A detailed description is given of the works and installations of this major Swiss development which includes the highest arch dam of the world. The Mauvoisin development utilises in two stages the waters of the Drance de Bagnes between Mauvoisin in the Upper Bagne Valley and Riddes in the Rhône Valley, and drains an area totalling 188 sq. km., of which 41% consist of glaciers. The first stage stretches from the Mauvoisin reservoir to the underground Fionnay power house, and the second from Fionnay to the surface plant at Riddes. The main features are as follows: The reservoir, 180 million cu. m. in capacity, 5 km. in length, covers an uninhabited area of 208 hectares and constitutes a reserve of 537 million kWh. There is an inclined surge tank, 319 m. long, 4 m. diameter, with expansion chamber at the top. The Fionnay underground power station operating at a head of from 47·4 m. to 30 m. at the rate of 34·5 cu. m. per sec., is equipped with three vertical 42,500 kW generating sets running at 750 r.p.m.—believed to be the highest head ever used with Francis turbines. In the second stage an intake has been constructed on the Dranse, upstream from Fionnay, to feed a 180,000 cu. m. balancing reservoir made of reinforced concrete and designed for a level variation of 12·5 m. Also a pressure tunnel 14,720 m. long and from 3·25 to 3·10 m. in diameter has been constructed and a surge tank 76 m. long, 3·1 m. dia., with double expansion chamber. A double penstock 1,760 m. long and from 1·7 to 1·5 m. in diameter, leads to the Riddes power house, which is equipped with five 67,000 kVA alternators each mounted between two single-nozzle Pelton wheels. A double tailrace 1,200 m. long returns the water to the Rhône.

The main characteristics of the Mauvoisin dam are: maximum height above foundations, 237 m.; total length at crest, 520 m.; width, 14 m. at crest, 53·5 m. at bottom. Elevation at crest, 1.962 m.; maximum elevation of water level, 1.961·5 m. Total volume of concrete placed, 2·03 million cu. m. Excavation work in connection with the dam alone amounted to 1·41 million cu. m., of which no less than 450,000 cu. m. was in the rock. The extent of this rock excavation work and the extreme steepness of the valley slopes at the dam site, combined with the necessity of building the cableway tracks at the same time, led to the adoption of a procedure which so far, though quite current in large-scale mining, had never been applied to dam building. In a first blasting operation, carried out on November 11, 1953, 20 tons of explosives packed in an aggregate length of 4,500 m. of drill holes displaced 66,000 cu. m. of rock. On

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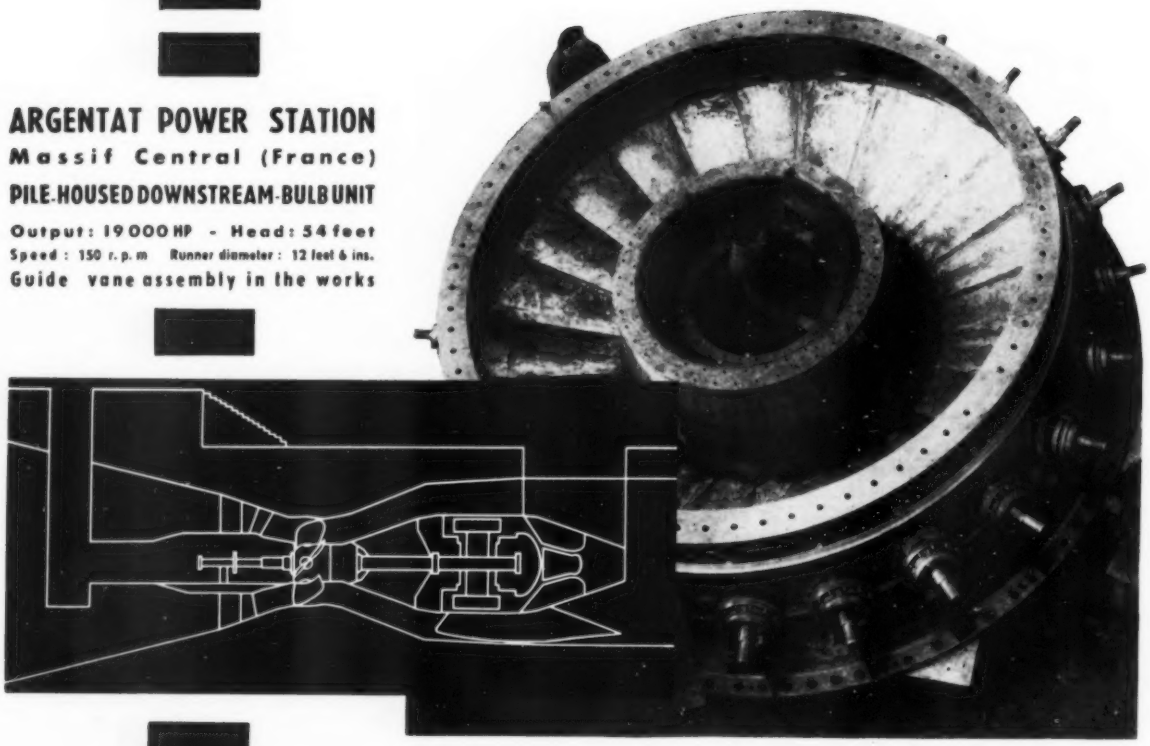
PILE-HOUSED DOWNSTREAM-BULB UNIT
Output: 19000 HP - Head: 54 feet
Speed: 150 r.p.m Runner diameter: 12 feet 6 ins.
Guide vane assembly in the works



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April 6, 1954, a second "grand minage" on an even larger scale (6,000 m. of drill holes, packed with 32 tons of explosives) disrupted 100,000 cu. m. of rock. In both instances, non-stop working power shovels cleared the excavation pits of the debris in a matter of three weeks. The construction plant, the preparation of aggregates and concrete, and concreting itself are described at length. Another major job of the contract was the driving of the pressure tunnels. The driving of the longest section (without adits) between Lourtiers and Isérables took four years and its concreting one more year. It is 10,395 m. in length. On August 6, 1956, after six years of unremitting efforts, the first set of the Fionnay station began to feed the French and Swiss grids. From then on, 770 million kWh have been supplied to Electricité de France and the six Swiss concerns which founded the Forces Motrices de Mauvoisin, S.A., Sion. (O. Rambert and M. Gavard, *Le Génie Civil*, Vol. 135, No. 9, May 1, 1958, p. 193, 13 pp., 26 ff.)

Clearing a Silted Reservoir

A description is given of the procedure applied to the clearing of 250,000 cu. yards of silt and 23,600 cu. yards of trees which, after a devastating flood caused by a thunderstorm in 1955, almost completely filled Cat Creek reservoir, a U.S. Navy Depot's water storage facility supplying the Naval Ammunition Depot at Hawthorne, Nevada. This flood scoured the canyon above Cat Creek dam virtually clean of overburden, trees and vegetation. A 500 g.p.m. sump pump, drawing from behind a small dike upstream of the dam and discharging through a 6 in. pipe over the crest of the dam, was first installed to keep the reservoir area dewatered. Six months later, in mid-December 1955, about 35,000 cu. yards of muck had been removed when a further abnormal flow due to unseasonable rains refilled the reservoir with muck overnight. With an estimated 250 cusecs of flood water coming down the canyon, the Navy finally approved, early in January 1956, the only solution likely to keep the site adequately dewatered. A 4 × 4 ft. hole was blasted through the 21-ft. thick dam wall at the base. There was much speculation as to how the 80 ft. layer of muck above the hole would react. Actually, at first, when the plunger for the last shot was pushed, only the rock plug moved; seconds later, fine gravel and mud began oozing out but, a full half hour later, the mud plug shot out with a roar and, that afternoon, 10 million gal. or about 50,000 cu. yards of water and mud raced down the canyon for several miles. It was now an easy matter for the muck to be flushed out. Muck removal then became a sequence of plugging the hole with the aid of a stiffleg derrick fitted with a 3½ cu. yard clamshell, allowing water level to rise, and finally reopening the hole with small charges of dynamite. This filling and flushing phase, during which actual flushing removed very little muck, was terminated in March. All told, about 40,000 cu. yards had then been cleared, not including the material carried away in the spectacular first flush. Thereafter, big bulldozers and pans were brought in to aid a Sauerman scraper in shifting the muck to the stiffleg derrick, which had been rerigged to deal with heavier loads. The clam shell on the stiffleg piled the material below the dam, and a power shovel, bulldozers and pans moved it down the canyon, forming a roadway and re-forming the watercourse. By June, the temporary 4 × 4 ft. outlet had been closed by means of a temporary stop gate of 1 in. steel plate welded to rail sections, and the reservoir was ready to resume service. Late last year, the rock and silt removal was completed by installing a hydraulically operated 36 in. penstock and a sluice gate in the 4 × 4 ft. hole. Under normal conditions, it is expected these measures will prove efficient, but the question as to what will happen in the case of another flash flood like that in 1955, is still unanswered. (Cdr. C. F. Mobley, U.S.N., *Engineering News-Record*, May 15, 1958, p. 40, 3 pp., 5 ff.)

Costa Rica Developments

It appears from this short survey that the central region of Costa Rica, which includes San José, the capital, has abundant power-water resources and, with a population of 580,000, also constitutes the main centre of consumption. According to the figures given by the Instituto Costaricense de Electricidad, consumption in 1955 was estimated at 250 million kWh. The total installed generating capacity is 68,650 kW, of which the 21 hydro-plants now in operation account for 43,850 kW. Most of these plants are on the small side and only nine exceed 1,000 kW. The largest is the Ventanas station, with 10,000 kW. A 30,000 kW project, designed for a future increase to 60,000 kW, is now nearing completion on the Rio Grande, and even larger plants are being planned on Rio Macho and Rio Reventazon. The I.C.E. expects the aggregate peak capacity to reach 139,000 kW by 1965 for the Central Region alone. (Jean Burz, *Die Wasserwirtschaft*, Vol. 48, No. 9, June 1958, p. 236.)

Concrete Placing in Large Dams

The author reviews the problems which confronted the contractors of the great Mauvoisin dam in connection with concrete placing. The first major improvement, suggested by Professor Stucky, was the use of bulldozers to spread the heaps of concrete poured by 6 cu. m. skips, so that the 70 cm. long pokers of the vibrators penetrated much more easily into the freshly poured and flattened layers. Compaction gained in quality, but a difficulty still remained in that the worker was unable to control efficiently the 75 to 80 kg. vibrator in the concrete mass; in addition, the distance between neighbouring vibrators varied too widely and was left too much to the workers' discretion. The problem was finally solved by mechanising the vibration process by fitting sets of four vibrators at the front end of specially designed tractors. Uniformity was thus achieved both in the spacing of the pokers and their inclination, while the depth of penetration of the tools into the concrete could be set as required. Further advantages of the method are: possibility of increasing the number of vibrators to six or eight per tractor, according to the volume of concrete placed at a time; easier checking of the actual vibration period, since the machine is not affected by the worker's fatigue; easy regulation of the backing of the battery of vibrators and the depth of penetration of the pokers; finally, especially at sites of high altitude, the machine is much less sensitive to weather effects than men. All these factors combine to bring about a uniformity of quality in the concrete which would be impossible with hand

vibrating. This has been amply established by 50 cm. dia. control borings carried out at various points of the dam to the extent of 150 m. in aggregate length. Sample cores of concrete of exactly the same composition obtained before and after the introduction of the new method showed that mechanically vibrated concrete was practically free from air bubbles, while the air content of hand vibrated concrete was still fairly substantial. This procedure, which enabled the Mauvoisin contractors to place as much as 8,000 cu. m. of concrete in a working day, has also been adopted with success at the Gougra Dam. (A. Coudray, Electrowatt Zurich, *Bulletin Technique de la Suisse Romande*, Vol. 83, No. 19, September 14, 1957, p. 319, 4 pp., 8 ff.)

Scour Protection

In this paper, read before the meeting of the Société Hydrotechnique de France in Paris in June 1957, the author reports on the laboratory investigations carried out in connection with the Cadarache dam of the Durance river development. Proceeding from general theoretical considerations, he examines the efficiency of a horizontal apron, the simplest structure of its kind, as the initial factor of an energy dissipating hydraulic jump, and suggests a method of calculation for such a structure with a view to maintaining the position of the hydraulic jump at the toe of the dam. The model tests are described. In the instance of a multi-sluice spillway, the author stresses the importance of operating on a model which reproduces in the laboratory channel not only a complete sluice but also part of the adjacent passes. The patterns adopted for the Cadarache dam tests are discussed, the longitudinal profile of the apron, its roughness, the influence of various shapes of piers, and protection by a rubblestone sill, being treated in separate paragraphs. A graphical method is suggested to determine the best operating methods for multi-sluice dams or spillways. In conclusion, the paper emphasises the general character of the investigations and the possibility they offer of restricting trial and error procedure in such an empirical sphere as the study of scour protection below dams or spillways. An account of the discussion which followed the lecture is appended. (B. Fauconnier, Ingénieur EDF, *La Houille Blanche*, Vol. 13, No. 2, March–April 1958, p. 123, 22 pp., 14 ff.)

Tingambato Project

The 156 MW underground power station at Tingambato constitutes the sixth stage of the great Miguel Alemán development of the Mexican Comisión Federal de Electricidad (CEF), one of the greatest hydro projects of Central America. Sited about 200 km. from Mexico City, Tingambato utilises the discharge of the Santa Barbara plant, which, together with the waters of the Ixtapan del Oro and Tilostoc rivers flows into the Santo Tomas reservoir, 5 million cu m. in capacity. A pressure tunnel, 10,140 m. long and 3·6 m. in diameter, leads to the 3·2 million cu. m. Los Pinzanes reservoir. A further pressure tunnel, 4,230 m. long, 3·7 m. in diameter, connects downstream with a surge tank and subsequently with a 375 m. vertical steel penstock erected in a cement-lined shaft 4·2 m. in diameter. The diameter of the penstock, which is closed at the top by a 3·48 m. in diameter butterfiy valve, varies between 3·46 and 3·32 m. The machine hall proper measures 63 × 12 m. and is 17·8 m. high from the hall floor. A 550 m. tunnel, with a 12% gradient, gives access to the machine hall. The three vertical 72,000 h.p. Francis turbines operate at a head varying between 362 and 394 m. and are designed to discharge 15·7 cu. m. per second. Their main characteristic is that they can run at either 600 r.p.m. (50 c.p.s.) or 720 r.p.m. (60 c.p.s.), thus enabling the alternators to feed either the metropolitan or the provincial grid. At the higher speed, it is necessary to change the runner of the turbines in order to maintain the efficiency of the turbines. The 57,500 kVA synchronous alternators manufactured and erected by Tecnomasio Italiano Brown Boveri, Milan, are described in detail. These heavy units are conspicuous by their uncommon axial length, and are designed for a runaway speed of 1,100 r.p.m. and a moment of inertia of 210 ton m². The rotors, each weighing 55 tons, had to be loaded on ships provided with adequate lifting gear, as the port of Vera Cruz has no cranes capable of handling such heavy pieces of machinery. The machine hall and penstock shaft alone entailed a volume of excavation of 32,000 cu. m. In addition, access to the various intake works required an aggregate length of 37 km. of roads, involving earthwork to the extent of about 2 million cu. m. (*Rassegna Tecnica Tecnomasio Italiano Brown Boveri*, Vol. XIX, No. 1, January-March 1958, p. 1, 9 pp., 9 ff.)

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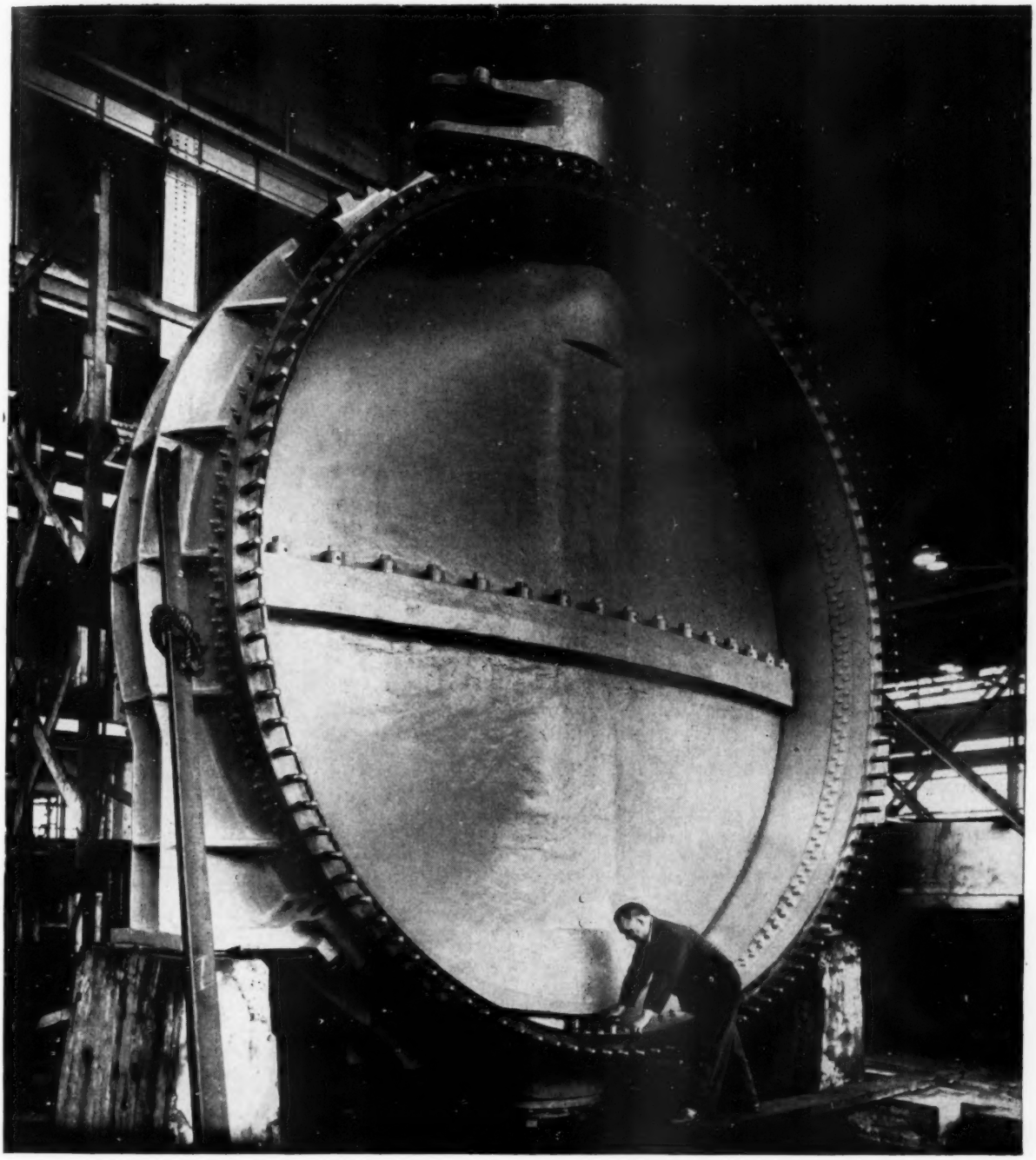

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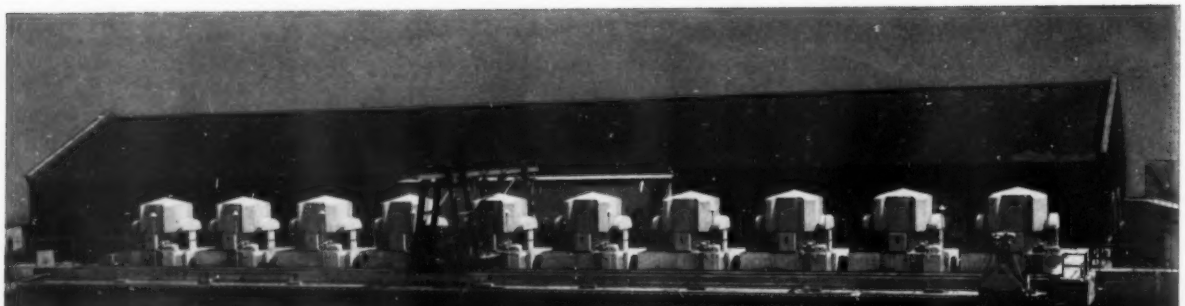

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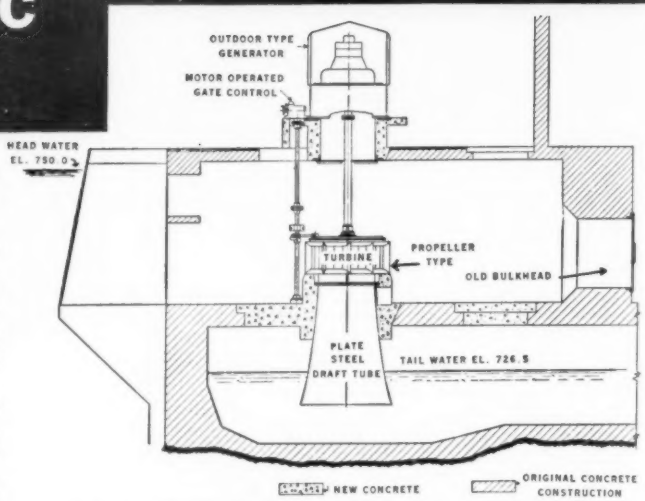

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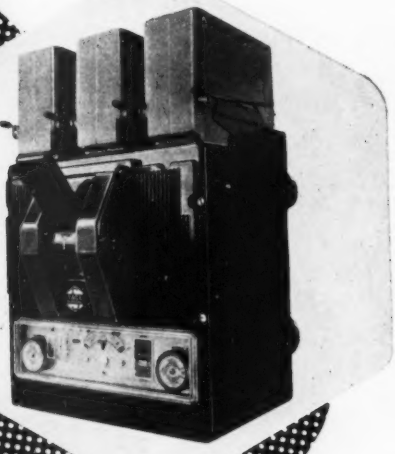

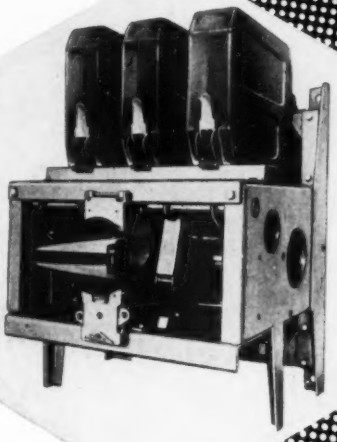

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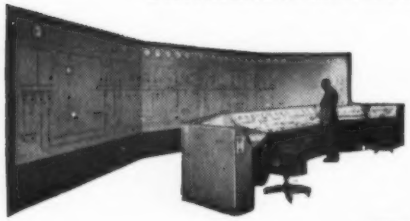

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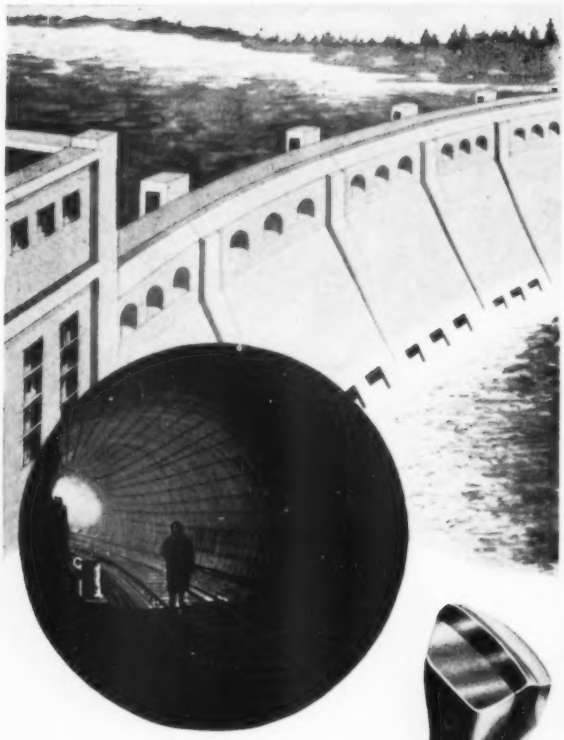

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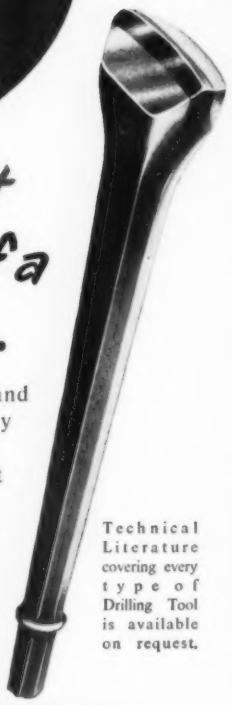

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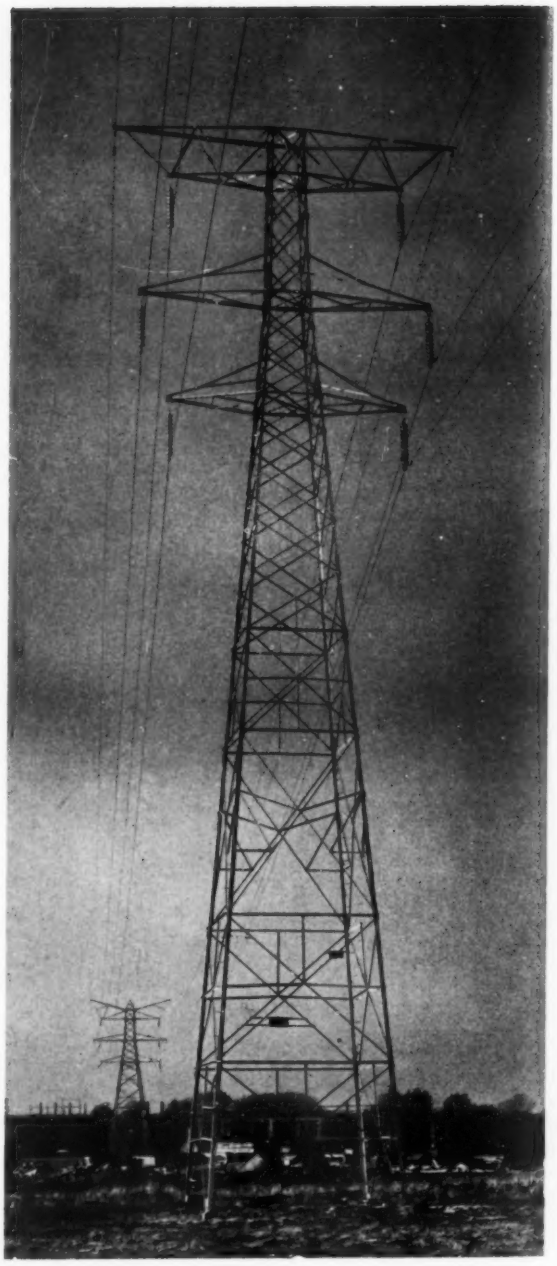

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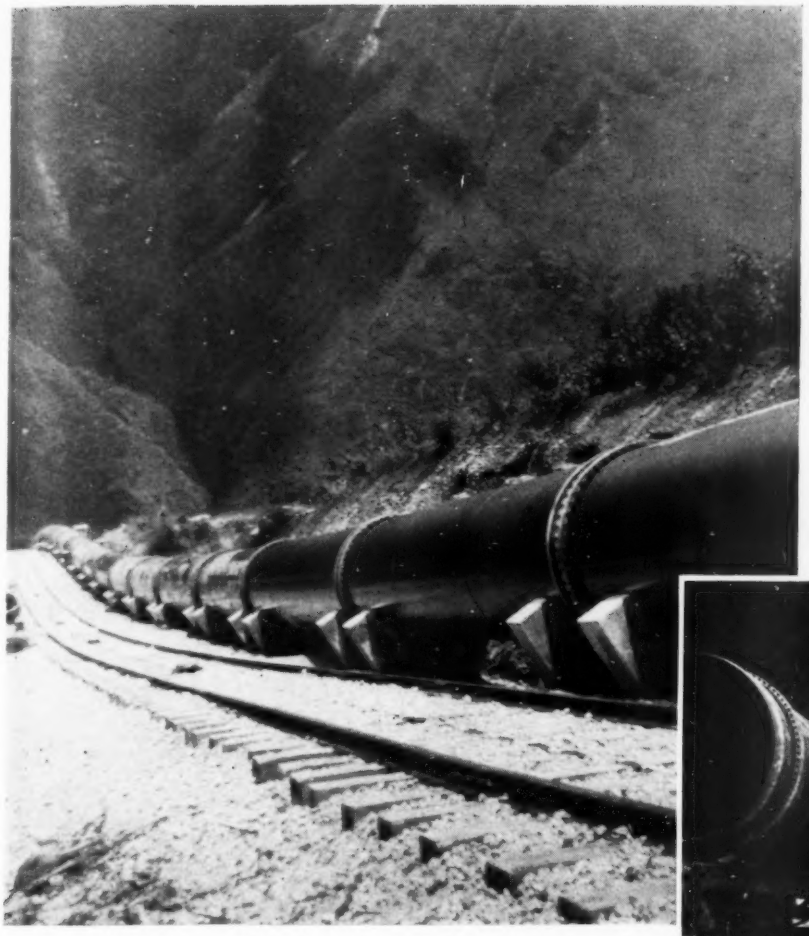

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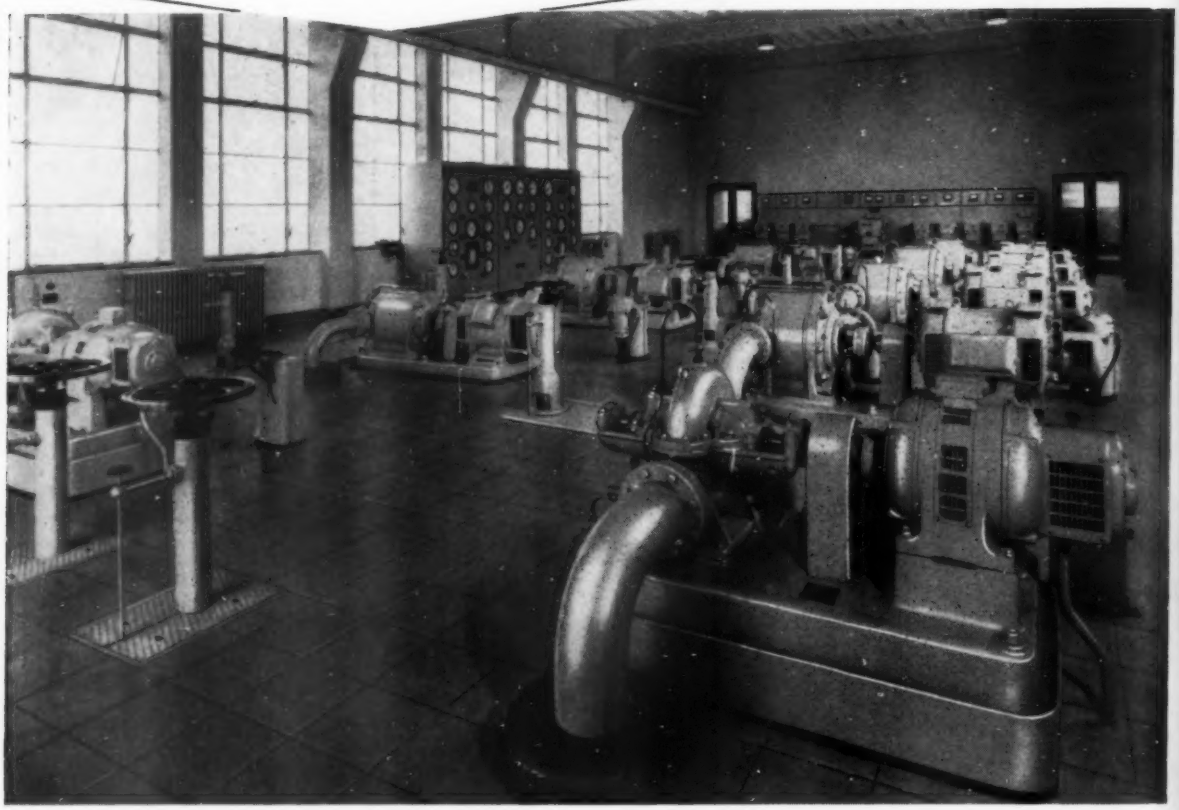

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